STANDARDS DEVELOPMENT BRANCH OMOE
3693600000 5942

SEWER AND WATERMAIN CONSTRUCTION INSPECTORS' COURSE

Municipal Engineers Association

Ontario Ministry of the Environment

E.T.C.

FLEMING
SIR SANDFORD FLEMING COLLEGE
CONFERENCE AND SEMINAR SERVICES

Ref. 72918

© 1978 HER MAJESTY THE QUEEN IN

RIGHT OF ONTARIO

SEWER AND WATERMAIN

CONSTRUCTION INSPECTORS COURSE

First Edition:	December 1978
Second Edition:	December 1979
Third Edition:	November 1980
Fourth Edition:	September 1985
Fifth Edition:	December 1989
Sixth Edition:	September 1990
Seventh Edition:	January 1993

Training & Certification Section
Human Resources Branch
Ministry of the Environment
135 St. Clair Avenue West
Toronto, Ontario M4V 1P5

17828

Copyright Provisions and Restrictions on Copying:

This Ontario Ministry of the Environment work is protected by Crown copyright (unless otherwise indicated), which is held by the Queen's Printer for Ontario. It may be reproduced for non-commercial purposes if credit is given and Crown copyright is acknowledged.

It may not be reproduced, in all or in part, part, for any commercial purpose except under a licence from the Queen's Printer for Ontario.

For information on reproducing Government of Ontario works, please contact Service Ontario Publications at copyright@ontario.ca

CURRICULUM DEVELOPMENT COMMITTEE

Roger Crawford, P.Eng., Chair	Ministry of the Environment
Mike Damiano, P.Eng.	Marshall Macklin Monaghan Limited
Robert Fudge, C.E.T.	Regional Municipality of Durham
Sing Lau, P.Eng.	City of Etobicoke
Cheryl Martin, BES	Ministry of the Environment
Ozzie Peel, P.Eng.	Proctor & Redfern Limited
Gail Swaine, P.E.	Gore & Storrie Limited
Doug Walker, P.Eng.	R.V. Anderson Associates Limited

ACKNOWLEDGEMENTS

The Curriculum Development Committee wishes to express appreciation to the following persons who have served on the Committee and/or contributed to the manual.

Gary Albert
Daryl Burns
Crystal Butler
Roger Clarke
John Croxall
Peter Fellowes
Dick Flaherty
Jack Garnett
William Gray
Chris Hinde
Peter Lyall
Andy Matwichuk
Lance McAnuff
Steve McKelive
Len Ostick
Harry Oussoren
Merv Perkins
Mike Perkins
Bill Roycroft
Murray Sarafinchin
Tim Sharp
Joe Tersigni
Cecil Wilson
Jack Windecker
Jerry Yaremovich
Jerry Yee

MOE
Standards Development Branch
LIBRARY
ENTERED JUN 2 6 2003

TABLE OF CONTENTS

ROLE OF THE INSPECTOR 1-1

RECORDS 2-1
- GENERAL 2-1
- OVERALL RECORDS 2-1
  - Preconstruction Records 2-2
  - Works Diary 2-2
  - Daily or Weekly Reports 2-3
  - Record of Quantities 2-6
  - As-Built Records 2-6
  - Pictorial Records 2-8
- SUMMARY 2-8

TENDERING PROCEDURES 3-1
- GENERAL 3-1
- PRE-TENDERING 3-1
- TENDERING PERIOD 3-2
- TENDER OPENING 3-4
- TYPES OF CONTRACTS FOR TENDERING 3-6
- ADDITIONAL TENDERING COMMENTS 3-10

BASIC ELEMENTS OF CONTRACT DOCUMENTS 4-1
- GENERAL 4-1
- THE CONTRACT 4-1
- TYPES OF CONTRACTS 4-2
  - Lump Sum Contract 4-2
  - Unit Price Contract 4-4
  - Cost Plus Contracts - General 4-4
- CONTRACT DOCUMENTS 4-6
  - Agreement 4-7
  - General Conditions of Contract 4-7
  - Specifications 4-10
  - Special Conditions 4-12
  - Contract Drawings 4-12
  - Form of Tender 4-12
  - Information of Tenderers 4-14
  - Addenda 4-14
  - Supplementary General Conditions
  - Supplementary Specifications 4-14
- CONFLICTS 4-14
- DISCHARGE OF CONTRACTS 4-16
- CONCLUSION 4-18

ENVIRONMENTAL CONSIDERATIONS 5-1
- GENERAL 5-1
- ENVIRONMENTAL ASSESSMENT ACT, 1975 5-1
  - Purpose 5-1
  - Who is subject to the Act? 5-3
- ENVIRONMENTAL ASSESSMENT TYPES 5-3
- ENVIRONMENTAL CONTROL PROCEDURES 5-6

Construction Equipment Maintenance 5-6
Slippage Areas 5-7
Right-of-Way 5-7
Clearing of Vegetation 5-8
Stockpiling of Excavated Material 5-8
Sediment Control and Erosion 5-9
Placement of Soil in Woodlots 5-9
Stream Crossing 5-10
Discharge to a Watercourse 5-10
Groundwater 5-10
Dust Control 5-11
Hydrostatic Testing 5-11
Restoration 5-11
Conclusions 5-12

CONSTRUCTION SAFETY 6-1
INTRODUCTION 6-1
THE LAW 6-1
CONTRACT DOCUMENTS 6-2
Inspector and Inspection 6-2
Public Convenience and Safety 6-2
The Occupational Health and Safety Act (Chapter 321 amended) 6-3
Ontario Regulation 213/91 (Construction Projects) 6-5
Part I General (Sections 1-19) 6-5
Part II General Construction (Sections 20-221) 6-5
Part III Excavations (Sections 222-242) 6-8
Part IV Tunnels, Shafts, Caissons & Cofferdams (Sections 243-331) 6-10
Part V Work in Compressed Air (Sections 332-397) 6-10
GENERAL 6-11

GUIDELINES FOR TRAFFIC CONTROL IN A CONSTRUCTION AREA 7-1
GENERAL 7-1
PRE-CONSTRUCTION PLANNING 7-3
CONSTRUCTION INSPECTOR'S ROLE IN CONTROL OF
TRAFFIC IN A CONSTRUCTION AREA 7-6
APPENDIX 'A' 7-9

SANITARY AND STORM SEWER DESIGN 8-1
GENERAL ASPECTS OF SANITARY SEWERS 8-1
Area to be Serviced 8-1
Land Use 8-1
Population Prediction 8-3
DETAILED ANALYSIS OF THE AREA 8-4
Analysis of the Area 8-4
Ground Cover 8-4
Soil Conditions 8-4
Population Prediction Methods 8-6
Distribution of Population 8-6
FLOW PREDICTION 8-7
Sewage Flows 8-7
Common Expedient 8-8
ACTUAL DESIGN OF SEWERS AND APPURTENANCES 8-8
Design Period 8-8

Design Flows 8-8
Infiltration 8-11
Grade 8-11
Size of Sewer 8-13
PUMPING STATIONs 8-14
Requirements for a Pumping Stations 8-14
Forcemain 8-15
Inverted Syphon 8-15
Manholes 8-16
House Service Connections 8-16
The Design Flow 8-24
Procedures for Estimating Run-off 8-26
The Rational Method 8-26
Expected Flow - "Q" 8-27
Run-off Co-efficient - "R" 8-27
Stormwater Management 8-29
Stormwater Quality 8-29
Protection Against Basement Flooding 8-30
Catch Basins 8-30
Construction 8-30

WATERMAIN DESIGN 9-1
GENERAL 9-1
WATERMAIN DESIGN 9-1
Flow Requirements 9-1
Hydraulic Design 9-8
Selection of Pipe Materials 9-15

GEOTECHNICAL CONSIDERATIONS IN SEWER & WATERMAIN CONSTRUCTION 10-1
INTRODUCTION 10-1
GLACIAL GEOLOGY 10-2
Glaciers 10-3
Deposition 10-5
SUBSURFACE INVESTIGATION 10-7
Field Testing Methods 10-9
SOIL CLASSIFICATION 10-12
Visual Classification 10-12
Laboratory Tests 10-14
Imported Granular Fills 10-15
COMPACTION 10-16
Definition 10-16
Compaction Testing 10-17
Factors Influencing Compaction 10-21
Validity of Insitu Density Measurement 10-22
Compaction Equipment 10-24
Frequently Encountered Compaction Problems 10-24
TRENCH EXCAVATION 10-27
Trench Excavation Problems 10-27
Trench Stabilization 10-29
Trench Backfilling 10-29
Geosynthetics 10-35
GROUNDWATER 10-36

Groundwater Control Methods 10-38
FOUNDATIONS 10-40
Shallow Foundations 10-40
Deep Foundations 10-41

PRACTICAL ASPECTS OF DEWATERING 11-1
INTRODUCTION 11-1
VACUUM WELLPOINTS 11-1
DEEP WELLS 11-7
EDUCTOR SYSTEMS 11-9
PRACTICAL CONSIDERATIONS 11-11
COST 11-18
RESPONSIBILITY 11-19
MONITORING OF LOCAL DOMESTIC WELLS 11-19
CONCLUSIONS 11-20

BLASTING CONTROL 12-1
LEGISLATION 12-1
Federal 12-1
Provincial 12-1
Municipal 12-2
CONTROL PROCEDURES 12-3
Pre-Blast Surveys 12-3
Monitoring and Criteria 12-5
Impact Identification 12-6
BLAST OVERPRESSURE 12-11
Effects on Water Supply 12-16
Determining Attenuation 12-18
Effects of Patterns and Procedures 12-18
CONTROL SPECIFICATIONS 12-20

BLASTING TECHNIQUE 12-21
EXPLOSIVE TYPES 12-21
Initiation Systems 12-21
BLASTING SAFETY 12-24

TUNNELLING 13-1
INTRODUCTION 13-1
Glossary 13-1
METHODS OF TUNNELLING 13-1
Shafts 13-2
Hand Mining 13-3
Mining Machines and Shields 13-5
Pipe Jacking 13-6
Augering 13-8
Rock Tunnels 13-8
PUBLIC RELATIONS 13-9
INSPECTOR'S DUTIES 13-9
Hand Mining 13-10
Machine Mining 13-10
Pipe Jacking 13-10
Safety 13-11
Surveying 13-11

SANITARY AND STORM SEWERS 14-1
GENERAL ASPECTS 14-1
GRADE AND ALIGNMENT 14-1
MANHOLES 14-3
COMPLEX SEWER STRUCTURES 14-4
HOUSE SERVICE CONNECTIONS 14-6
CATCH BASINS 14-6
MATERIALS USED IN SEWER CONSTRUCTION 14-7
CONSTRUCTION CONSIDERATIONS 14-8

WATERMAINS 15-1
GENERAL ASPECTS 15-1
SYSTEM LAYOUT AND ALIGNMENT 15-1
WATERMAIN APPURTENANCES 15-5
Control Valves 15-5
Air Relief Chambers 15-6
Drain Valves 15-6
Valve Chambers 15-6
Valve Boxes and Stems 15-8
Hydrant 15-8
Service Connections 15-8
Bends, Wyes and Tees 15-9
Thrust Blocks 15-9
BEDDING 15-11
MATERIALS USED IN WATERMAIN CONSTRUCTION 15-11
CONSTRUCTION CONSIDERATIONS 15-12

CONSTRUCTION SURVEYING 16-1
INTRODUCTION 16-1
CONSTRUCTION SURVEYS 16-4
Preparation For Surveying 16-5
Construction Layout 16-7
Construction Grades 16-11
GRADE SHEETS 16-12
Cut Determination 16-13
Grade Transfer 16-15
Alternative Methods of Grade Transfer 16-22
House Connections 16-23
Checking of Layout 16-23
QUANTITY CALCULATIONS 16-25

APPENDIX - 16A 16A-1
GENERAL SURVEYING 16A-1
REQUIRED MEASUREMENTS 16A-2
SURVEYING EQUIPMENT 16A-2

APPENDIX - 16B 16B-1
CHAINING 16B-1
Corrections to Field Measurements 16B-1
Good Chaining Practices 16B-3
Accuracy 16B-3

APPENDIX - 16C 16C-1

LEVELLING 16C-1
The Level 16C-1
Adjusting the Level 16C-1
Three Peg Test 16C-2
Theory of Levelling 16C-2
Accuracy 16C-8

APPENDIX - 16D 16D-1
ANGULAR MEASUREMENT BY TRANSIT 16D-1
Measurement of a Horizontal Angle 16D-2
The Telescope Sight 16D-2
Verniers 16D-3
Adjustment of the Transit 16D-3
Care of the Instrument 16D-6
BEARINGS AND AZIMUTHS 16D-8

APPENDIX - 16E 16E-1
INTRODUCTION 16E-1
TRADITIONAL METHODS FOR PROVIDING LINE & GRADE CONTROL 16E-1
SUMMARY 16E-3
THE USE OF LASERS FOR PROVIDING LINE AND GRADE CONTROL 16E-5
APPLICATIONS 16E-11
Most Common Set-Ups 16E-15
MODEL 932 TRIVET PLATE ON A MANHOLE FLOOR 16E-16
MODEL 932 TRIVET PLATE FOR LARGE PIPE 16E-17
MANHOLE WITH POURED INVERT OR RAISED INVERT 16E-19
MANHOLE WITH SOLID BOTTOM 16E-21
CROSSARM BRACES IN LARGE PIPE 16E-24
HI-LINE ACCESSORY SETUP PROCEDURE 16E-26
MODEL 607 MANHOLE MOUNT WITH 966 HI-LINE SIGHTING SCOPE & ADAPTOR 16E-30
OPEN EXCAVATION WITH A TRIPOD 16E-33
HI-LINE ACCESSORY SETUP PROCEDURE - FOR TRIPOD SET-UP 16E-34
SUBJECT OF SPECIAL INTEREST TO USERS OF SEWER LASERS 16E-38
Refraction 16E-38
Calibration 16E-41
SAMPLE SPECIFICATIONS FOR LASER CONTROL ON SEWER CONSTRUCTION ... 16E-43
Line and Grade Control 16E-43
Line and Grade Control 16E-44
SUMMARY 16E-45

EXCAVATION AND BACKFILL 17-1
Pre-Excavation 17-1
EXCAVATING EQUIPMENT 17-2
SOILS JUDGEMENT 17-4
STORAGE OF EXCAVATED MATERIAL 17-5
EXCAVATION SHAPE 17-5
BACKFILL EQUIPMENT 17-9
BACKFILL PROBLEMS 17-11
Approaching the Surface 17-12
Surplus Excavated Material 17-12

PIPE INSTALLATION 18-1
GENERAL 18-1

HANDLING 18-1
Concrete Pipe 18-1
Ductile Iron Pipe 18-1
Polyvinyl Chloride and/or Polyethylene Pipe 18-2
Vitrified Clay Pipe 18-2
Corrugated Steel Pipe 18-2
STORAGE 18-2
Gasket Storage 18-3
JOINTING 18-4
General 18-4
Concrete Sewer Pipe 18-4
Vitrified Clay Sewer Pipe 18-4
Ductile Iron Pipe 18-5
Polyvinyl Chloride Sewer and Pressure Pipe 18-7
Polyethylene Pipe 18-7
Principal Considerations for Jointing 18-10
BEDDING 18-10
General 18-10
Classes of Bedding 18-12
Class 'A' Bedding 18-12
Class 'B' Bedding 18-12
Class 'C' Bedding 18-12
Class 'D' Bedding 18-12
Bedding Procedure 18-15
Bedding Material 18-15
Principal Considerations for Bedding 18-16

DISINFECTION OF WATERMAINS 19-1
GENERAL 19-1
CLEANING 19-1
DISINFECTION 19-3
Gas Chlorination 19-3
Hypochlorination 19-3
APPLICATION PROCEDURES 19-4
Continuous Feed 19-4
Slug Method 19-6
Flushing 19-6
BACTERIOLOGICAL TESTING 19-7

LEAKAGE AND PRESSURE TESTING 20-1
LEAKAGE TESTING OF SEWERS 20-1
Infiltration Test 20-2
Exfiltration Test 20-5
General Notes for Testing 20-8
Low Pressure Air Testing 20-8
Test Procedure 20-9
Safety Requirements 20-13
General Comments 20-13
Ground Water Adjustment 20-15
LEAK LOCATION IN NEW SEWERS 20-15
ALIGNMENT CHECK OF SEWERS 20-17
TESTING OF WATERMAINS 20-17
TESTING OF SEWER FORCEMAINS 20-21

RESTORATION 21-1
General 21-1
Inspector Responsibilities 21-2
Restoration Considerations 21-3

SEWER REHABILITATION 22-1
INTRODUCTION 22-1
SEWER DISTRESS 22-1
SEWER INSPECTION 22-2
METHODS OF SEWER REHABILITATION 22-4
SUMMARY 22-15

WATERMAIN REHABILITATION 23-1
INTRODUCTION 23-1
REHABILITATION - BY CLEANING 23-3
REHABILITATION - BY RELINING 23-3
Cement Lining 23-3
Slip-Lining 23-6
Chemical Lining 23-6
Chemical Film 23-8
Chemical Coating 23-8
Inversion (insituform) Lining 23-9
CATHODIC PROTECTION 23-10
REPLACEMENT OF WATERMAINS 23-11
SERVICE CONNECTIONS 23-12

SUBJECT:

CONSTRUCTION OF SEWERS
WATERMAINS AND ASSOCIATED
FACILITIES

TOPIC: 1

ROLE OF THE INSPECTOR

OBJECTIVES:

The trainee will be able to:

1. Understand the role of the inspector concerning:

   a) duties to the owner and contractor;

   b) maintenance of public relations in the Municipality.

ROLE OF THE INSPECTOR

The importance of the inspector on a construction project cannot be over-emphasized. He is the day to day link between the owner and the consulting engineer on the one hand and the contractor on the other. Although the main duty of the inspector is to protect the interests of the owner, by seeing that the work contracted for is carried out. In carrying out that duty he must be fair to the contractor. The inspector should be polite in his dealings with others but firm in his insistence that the contractor construct the works properly and in accordance with the contract.

Most contracts make the contractor responsible for complying with all applicable bylaws and safety regulations and for carrying out the works in such a manner as will not cause unreasonable or unnecessary inconvenience to the public. It is one of the duties of an inspector to check that the contractor complies with these requirements and when necessary to remind him and even (if the contract allows) order him to comply.

The inspector is often the last man who has the opportunity to spot mistakes such as dimensional discrepancies or deterioration of soil conditions before the affected part of the works has been constructed and any necessary corrections have become an expensive item. The inspector should not make corrections or modifications to the work himself and, should refer such matters to his head office and obtain directions from those responsible for the design of the works. Written memos are required.

The method of construction and the construction plant to be used are generally the contractor's choice and responsibility (except when, because of a special design or particular soil conditions, the contract specifies the method or plant to be used). There will be times, however, when it becomes apparent to the inspector that the method or plant being used by the contractor is unsuitable for the job in hand. The inspector will maintain a complete set of notes and reference his office as per the daily report.

In extreme cases, if the method or plant being used is likely to be a danger to people, property or the works, the inspector may have to order the contractor to stop work or change his method and such action shall be followed immediately with consultation by the inspector with his head office.

The inspector should try to spot sub-standard materials and workmanship as early as possible and point them out to the contractor immediately so that the amount of unsatisfactory work requiring taking out and re-doing is kept to a minimum. All material is to be pre-checked prior to installation.

An important aspect of an inspector's job is the maintenance of good public relations in the municipality. If the inspector ensures that the contractor gives the local authorities, property owners and tenants notice of proposed interruption of existing services, temporary blocking of streets, blasting operations, etc., he will find that people are generally more co-operative and more willing to put up with such inconveniences. The period of inconvenience will be shortened and hence public relations improved if clean-up follows closely behind construction on streets and easements. Few contractors will carry out restoration and clean-up promptly of their own accord as they tend to look upon such work as a non-paying item. The public, on the other hand, tends to judge a sewer or watermain job by what is visible above ground and usually, after backfilling has been completed, all that is visible is restoration and clean-up. The inspector must, therefore, keep close watch on such work and insist that the contractor carries it out promptly.

In dealing with the public, the inspector must be careful not to make rash promises. He cannot commit the contractor to work which is not required by the contract and he must not commit the owner to additional work without first having the matter approved by the owner through the inspector's head office.

The responsibility and authority of an inspector is normally set out in the General Conditions of Contract. The Inspector should be familiarized with the General Conditions of any contract for which he/she is required to perform inspection duties. In the General Conditions, the object is to set forth both the general duties, authority, responsibility and relationships of the contracting parties and their representatives in the actual conduct of the work, whether in the office or in the field. General Conditions may vary.

SUBJECT:

CONSTRUCTION OF SEWERS WATERMAINS AND ASSOCIATED FACILITIES

TOPIC: 2

RECORDS

OBJECTIVES:

The trainee will be able to:

1. Understand the importance of accurate and detailed records.
2. Organize records of work progress.

RECORDS

GENERAL

One very important function of the inspector is to keep records as the job progresses. It is essential to record work progress, daily events, weather and soil conditions, location of rock, uncharted underground obstructions or services, failures of the contractor as well as any remedial action taken.

Records should be comprehensive and accurate so that they will stand up before any court of law. The inspector should always assume that his firm or municipality may be required to substantiate its action in court and keep his records accordingly. When litigation does take place a court will put greater reliance on reports, properly prepared at the time, dated and signed, rather than on a person's memory of events which took place many months or years before.

OVERALL RECORDS

Whereas record keeping practices vary from municipality to municipality or from employer to employer, the basic ideas presented below do not vary.

An inspector must be able to locate any installation done under his supervision at any time. He must also be able to compute and substantiate the quantities comprising the contract. To be able to do so, he must keep a fairly comprehensive set of records.

The following sections will outline some of the most common and basic records that should be kept.

Preconstruction Records

Preconstruction records are required to outline the existing conditions of pavement and ground surfaces, structures, streets, and shrubs along the routes the works will take. As is outlines in the topic "Restoration" this written and/or photographic record is vital to ensuring that proper restoration takes place. The record should be made prior to the start of construction and should outline any areas of possible contention in the future. Any photographs or video tapes taken should be labelled with date, location (by chainage number), project name and contract number.

Works Diary

The work diary should consist of a book with a hard cover and be kept up-to-date at all times. It should never be a loose leaf book since pages can be added or removed at a later date. All entries should be made in ink. Every daily page must be signed by the person making the entry and those daily entries generally should consist of:

1. date
2. weather conditions, high and low temperatures
3. size of work force
4. equipment on site
5. location where contractor worked
6. progress of works
7. special events, accidents, etc.
8. orders given to the contractor
9. site meetings
10. visitors to the site
11. instructions received from supervisors
12. description of soil conditions encountered
13. any equipment or materials delivered to the works for incorporation in the works
14. comments entered by the contractor
15. any other relevant information or comments

Daily of Weekly Reports

Daily or weekly reports should be prepared in triplicate as required. One copy should be kept by the inspector for his personal records.

The contents of the daily report should be the source for the daily entries in the diary.

The advantage of the daily or weekly report is that the inspector's supervisors and the owner are kept informed of the progress of the work as well as the special events and difficulties encountered on the site. Thus they are able to take action when required without undue delay.

Daily reports should include the following:

1. Project name, contractor's name, owner's name, project number, contract number.

2. Date, weather, temperature.

3. Day's progress (by chainages and lengths).
   - trenching completed
   - pipe installed (describe any "specials" installed)
   - bedding completed
   - backfilling completed
   - surface restoration completed
   - rock excavation.

4. Appurtenances constructed
   - maintenance holes (chainage and depth)
   - catch basins (chainage)
   - laterals (chainage and depth, size and length)
   - chambers (chainage)
   - valves (chainage)
   - fittings (chainage)

5. Size, depth and chainage location of structures encountered:
   - centre line of roads crossed
   - utilities and service connections
   - underground chambers and structures.

6. Record:
   - type of soil encountered
   - extent of shoring used
   - ground water conditions
   - dewatering methods

7. List major items of equipment and number of men on job for the general contractor and sub-contractors separately.
   - list materials delivered to job and condition.

8. Hours worked.

9. General comments:
   a) report any unusual happenings or incidents such as accidents as well as visitors to the site;
   b) record the results of any tests performed.

10. Signature of inspector:
   - the inspector should sign each day's report.

The daily report may be summarized weekly to form the basis of a weekly report which would be sent to the head office.

Record of Quantities

This record should be kept in accordance with the Schedule of Items and Prices of the contract documents.

The inspector should use a field book for keeping a record of the exact measurement of contract items up-to-date as the work progresses. Items difficult to prove later such as fill concrete, etc. should be recorded with sketches showing where the measurement has been taken. It is advisable to have such sketches initialled by the contractor or his foreman to avoid arguments later. This is especially important when dealing with force account items.

Rock excavated should be shown on the profiles of the construction drawings to allow proper calculation of the actual quantities. Rock elevations should be recorded in the inspector's field notes as well as on the drawings. The owner's representative should be able to obtain the quantity of work completed to date from this record at any time.

As-Built Records

The importance of as-built records cannot be overly emphasized. The following items should be recorded on the as-built records:

- existing ground (if different than on the drawings)
- structures and utilities encountered
- location, elevation, grade and sizes of installed pipe and appurtenances
- locations, length, sizes and surface ties for laterals, tees, bends and stubs (surface ties to be referenced to permanent structures)
- elevation of rock surfaces encountered
- unusual ground conditions
- support and sheeting left in place, anchor blocks, etc.

All the above measurements should be made at the actual time of construction particularly if the item is to be buried.

Pictorial Records

The inspector should have a camera on the site. A polaroid camera would be most desirable because the picture can be marked and identified shortly after it is taken. Video cameras are gaining wide usage.

A pictorial record, if properly done, can avoid many arguments later and also support the inspector's positions in court if necessary.

A picture showing the conditions of a street or property before construction gives proof that the contractor did or did not reinstate original condition.

A picture taken of the excavated trench shows proof of underground conditions, of the contractor's safety practices, etc.

Any unexpected condition or faulty workmanship should be recorded by taking a picture. The price of the film in comparison to the project cost is very little and should not be a reason for taking an insufficient number of pictures.

Every picture should be identified with the following information:

1. Project and contract number.
2. Municipality.
3. Location (station and view).
4. Date.
5. Description.
6. Photo taken by.
7. Witnessed by (if possible).

SUMMARY

Records comprise a large part of an inspector's job. If this portion is not done correctly he can cost the owner he is representing many dollars by not adequately documenting items of contention as well as materials used.

SUBJECT:

CONSTRUCTION OF SEWERS
WATERMAINS AND ASSOCIATED
FACILITIES

TOPIC: 3

TENDERING PROCEDURES

OBJECTIVES:

The trainee will be able to:

1. Understand the fundamentals inherent in the preparation and submission of tenders.

2. Identify the different types of tenders and the reasons for their use.

TENDERING PROCEDURES

GENERAL

A tender is defined as an offer to do work upon certain terms.

The offer or "bid" is usually in writing but it may also be oral. It may also be informal and may be opened in private or in public as a response to public advertisement or private invitation.

Tenders may also be negotiated between an owner and a contractor on a bilateral basis without any apparent competition.

Each of the various types of tenders may be considered correct or appropriate in certain circumstances. The purpose of this discussion is to identify the procedures which are more or less general to all tenders, and then to discuss the various types of tenders and when they might be used. It will also be pointed out why certain precautions are taken and why deposit cheques or bonds are held until a formal contract is signed.

PRE-TENDERING

Prior to seeking tenders it is obvious that the design drawings and specifications must be completed so that the work is very accurately described. Generally speaking it can be stated that the best drawings and most complete specifications produce the lowest prices since the contractor is not forced to guess at the specific intent of the documents.

Prior to tendering it is also usual and desirable that the owner carry out a review of all items which should be settled prior to tendering, in addition to the actual drawings and specifications. Property, when needed, should be purchased and in clear title of the owner prior to commencement. All direct and indirect approvals should be in the owner's hands and copies should be made available to the inspector prior to construction.

The Ontario Ministry of the Environment utilizes a pretendering check list which includes over one hundred items in the contract including everything from the correct wording in the title drawing to the size of the site office, amount of insurance required, statements on workmen's rights, etc., etc.

The inspector cannot be expected to recite all of these requirements but should be aware that a thorough review is necessary immediately prior to tendering.

TENDERING PERIOD

When the date for calling tenders finally arrives, the inspector nominated to inspect the work should ideally be given a set of plans and specifications to commence his familiarization with the project. He should particularly try to be present when test holes are being dug and should be given any addenda to the contract which may be issued during the tender period.

Addenda are formal notices of additions, deletions or changes to the specifications or drawings which are issued by the owner or his consulting engineer during the tender period.

Such addenda often arise from questions asked by the contractors who may be seeking clarification or explanation of some point in doubt.

The inspector, however, should never, during construction, accept a contractor's contention that a certain modification was verbally approved by the engineer during the tendering period.

Also, during the tendering period, the inspector should never advise or assist any of the tenderers in any way. All questions should be directed to the engineer for reply.

The tendering period varies considerably. For a short length of sewer in an urban area with many active contractors, a tendering period of as little as one week could be sufficient. For the same sewer in a remote town a four week period may be necessary in order to reach an available contractor in a nearby town. For major pollution control plants requiring many sub-contractors, a two month period might be necessary.

During the tendering period for a public tender, it is often necessary to produce as many as fifty sets of drawings. The contractors are charged a fee for the documents to help defray the cost of printing and also to discourage persons who may be only casually interested in the project.

During the tendering period, most advertisements requesting tenders include the following words: "Lowest or any tender may not necessarily be accepted". This wording is legal warning to the bidders to protect the owner if all the quoted priced are too high or if the low bidder is considered to be potentially insolvent or incompetent.

Withdrawal of tenders prior to the actual closing time is legally allowed, but the specific arrangements vary from authority to authority.

However, every owner, whether in the private or public sector and every consultant should make every effort to be totally fair to the tenderers who invest considerable time and money in preparing bids. Tenders should only be rejected for very serious and well founded reasons.

A final comment on the time of tendering, however, is that the closing time is a strict condition of the contract. Tenders which are received late should be returned up-opened, since it could be possible that some revelation of a competitor's price, kept confidential until closing time, might result in a contrived or dis-honest low price.

TENDER OPENING

When spending public funds, it is usually required and is always the best practice to open tenders in the present of the "public". Primarily this includes the pertinent tendering contractors, representative government officials plus interested spectators.

Each tender is opened and the tendered price is read aloud to those present, and an acknowledgement is made of the accompanying certified cheque or bid bond.

The cheques are usually held by the owner, and the tenders are submitted to the engineer for review.

The deposit which accompanies the tender is necessary to insure that a contractor will be obliged to enter into a contract at the tendered price. If no money were involved, the contractor who is low could simply say that he made a mistake and then walk away. The deposit is usually set at 5% of the estimated amount of the contract in expectation that the low and second bidders will have a spread of less than 5%. If the low bidder is perhaps 10% low he may wish to leave his deposit behind rather than lose money on the job. This is legal, but creates a bad reputation which may do more harm in the long run.

The owner risks his reputation by holding the contractor's deposit cheques for an extended period and if this becomes an established practice, then the competitive contractors will simply fail to respond to future requests.

On private work such as sub-divisions, current practice is leading to more and more "tendering" which is not accompanied by cash deposits. This, unfortunately, leads to post-tender negotiating and price-cutting which is detrimental to the entire process, and decidedly unfair to the contractors.

TYPES OF CONTRACTS FOR TENDERING

There are an infinite number of types of contracts and an infinite number of variations but as a sample of those currently in use, the following may be listed:

a) **Lump Sum Contract**

In a lump sum contract, the Contractor undertakes to carry out the works for a fixed sum of money. This form of contract is used where the work is reasonably certain in character and extent. Under this contract, the onus is on the Contractor to work more efficiently, for whatever difference exists between the contract lump sum price paid to the Contractor and his actual costs represents his profit or loss. This type of contract is most common for buildings, pumping stations and treatment plants with many components, too numerous to itemize, but which are clearly identifiable from the drawings. Such projects are usually on a single site and all pertinent information is given i.e. soils report etc. In a lump sum contract, the Inspector has the responsibility to keep good records and to ensure that the work is carried out in accordance with the contract specifications.

b) **Unit Price Contract**

This type of contract incorporates a Schedule of Quantities in which all of the items of work are listed and priced on a unit price basis, to give a total contract price. This type is primarily used on sewer, watermain and roadwork projects and on excavation jobs where it is difficult to state the exact volume of earth and rock to be encountered. In this contract the Contractor is paid for whatever work he carried out and one can see that the final value of the contract can fluctuate significantly, depending on how accurately the original quantities were estimated.

The role of the Inspector in this type of contract is to ensure that all the work completed by the Contractor is properly measured and documented and that the work has been carried out in accordance with the drawings and specifications.

c) **Cost Plus Contract - General**

In this type of contract, the Contractor is paid the cost of his materials, equipment, labour and overhead, plus a profit. This is a very difficult type of contract to administer and requires an on-site bookkeeper as well as inspector. Under cost plus arrangements it is important to agree, beforehand, what costs will be considered as relevant. There are three types of cost plus contracts namely:

<u>Cost Plus Percentage Contract</u>

In a cost plus percentage contract, the Contractor is paid the actual cost of work, plus an agreed percentage of the actual or allowable cost to cover overhead, profit, etc. It is useful in an emergency, when there is insufficient time available to prepare a detailed scheme or when the work cannot be clearly identified because of uncertain conditions.

The major disadvantage of this form of contract is the lack of incentive for the Contractor to complete the work as quickly as possible or to try to reduce costs. While there is some assurance to the Owner that there are no extravagant profits being paid to the Contractor, it is necessary to closely monitor and control the Contractor's records relating to the cost of the work to ensure that proper accounting is carried out.

Cost Plus Fixed Sum Contract

In this form of contract, the Contractor is paid the actual cost of the work plus a fixed sum, which has been previously agreed upon and which does not fluctuate with the cost of the job. It is superior to the cost plus percentage contract, as there is some incentive to the Contractor to complete the work as speedily as possible, although once again, the work records should be carefully checked and monitored.

Cost Plus Fixed Sum, with Bonus or Penalty, Contract

In this type of contract, the Contractor is paid the actual cost of the work with a fixed sum previously agreed upon. In this case, however, if the Contractor is able to reduce the cost of the work below the original estimated costs, he receives an agreed percentage of the savings, while if he allows the cost to exceed the estimate, he is penalized by deduction from the agreed fixed sum, a percentage of the excess of the actual amount over the estimated cost. As this form of contract incorporates an incentive to carry out the work as quickly and cheaply as possible, it constitutes the best of the "cost plus" type of contract, from the Owner's point of view.

d) **Target Price Contract** (Project Management)

This is not really a tendered contract but is a budget figure established after discussion with a chosen contractor who is expected to have considerable input into the design as well as the construction. Revisions are made during construction to react to market changes in available materials or special products may be utilized in innovative ways to achieve cost savings.

This type of contract is becoming increasingly popular in the private sector but since direct competition is eliminated in favour of expediency, it is difficult to assess the true economics, if any, in this type of contract.

ADDITIONAL TENDERING COMMENTS

a) **Bid Depository**

To permit sub-contractors to be equally competitive in the broad market on buildings, plants and other complex projects, a system has been evolved by the contractors themselves known as a bid depository.

For instance, a plumbing contractor may obtain plans and prepare a price for the plumbing work on a pollution control plant. His price is then submitted to the bid depository a few days prior to the closing of the general contract.

The general contractors then review the plumbing prices in comparison with all of the others submitted. The general may select the low plumbing price or select an alternative which is slightly higher but with whom he feels he can work more productively.

In either case, all plumbers' prices are available to all of the generals and the process is fair in that intimidation of a little sub by a big general contractor, is lessened.

b) **Formal and Informal Tenders**

A tender is considered "formal" if it meets exactly all the requirements of the owner and is accompanied by a valid deposit cheque or bid bond. A tender may be declared informal if the cheque is not certified or the tenderer has altered the documents in any way.

SUBJECT:

CONSTRUCTION OF SEWERS WATERMAINS AND ASSOCIATED FACILITIES

TOPIC: 4

BASIC ELEMENTS OF CONTRACT DOCUMENTS

OBJECTIVES:

The trainee will be able to:

1. Appreciate the features of a contract.
2. Be aware of the different types of contracts.
3. Understand the basic elements of contract documents.
4. Realize the means to discharge a contract.

BASIC ELEMENTS OF CONTRACT DOCUMENTS

GENERAL

Normally the Owner, the party desiring to have a project constructed is not adequately qualified to develop and direct such work and thus will employ specialists and skilled advisors to plan and supervise the execution of the work within stipulated cost and time limits. This specialist is usually the Engineer and in cases of building construction, the Architect. The Engineer does not possess the plant and organization to undertake the actual work and therefore it is normally arranged that another party, called a Contractor, particularly skilled in construction operations, is vested with the actual execution of the work under the general supervision and direction of the Engineer.

To ensure that all aspects of the "arrangement" are precisely identified and clearly understood, the Owner and the Contractor enter a contractual agreement outlining all of the important aspects of the work to be done.

THE CONTRACT

In simple terms, a contract exists when one party makes an offer to undertake certain works, and another party accepts the offer. An acceptance of an offer may in certain cases be deemed to consummate the agreement even though the formal signing of the agreement has not taken place. Another way to describe a contract may be a promise or a set of promises which the law will enforce. The laws of contracts differ from other branches of the law in the respect that the parties to the contract to a great extent, make their own rules.

In addition to the requirement of an offer by one party and acceptance of the offer by another party, the contract is characterized by several other essentials. The persons entering into a contract must be of legal age and cognizant of what they are doing. Thus, a contract with an infant, an intoxicated or insane person would be rendered void, or at least voidable.

The parties entering into a contract must have a mutuality of mind, that is to say, they must understand each other as to exactly what is the work that is to be undertaken and that there are no undisclosed pertinent facts.

Finally, a contract between the parties must be legal, thus a party cannot contract itself into undertaking an illegal action. Normally there are two major categories of contracts:

1. Unsecured cost contracts, and
2. Secured cost contracts.

There are many breakdowns of the above two general categories but it is evident from the classification that the first category does to guarantee the Contractor that he will recover all amounts that he has expended on the works whereas the second does. Most Government agencies deal with contracts of the first category almost exclusively.

The unsecured cost contracts have two major forms currently in use: the Lump Sum Contract and the Unit Price Contract. These are discussed more fully in the following section.

TYPES OF CONTRACTS

There are several types of contracts which are commonly in use today, and the nature of the project, its location, the complexity of the work, and the proposed scheduling of the project, all influence the type of contract most suitable for a given project. The following are the more common types of contracts:

Lump Sum Contract

In a lump sum contract, the Contractor undertakes to carry out the works for a fixed sum of money. This form of contract is used where the work is reasonably certain in character and extent. Under this contract, the onus is on the Contractor to work more efficiently, for whatever difference exists between the contract lump sum price paid to the Contractor and his actual costs represents his profit or loss. This type of contract is most common for buildings, pumping stations and treatment plants with many components, too numerous to itemize, but which are clearly identifiable from the drawings. Such projects are usually on a single site and all pertinent information is given i.e. soils report etc.

In a lump sum contract, the Inspector has the responsibility to keep good records and to ensure that the work is carried out in accordance with the contract specifications.

Unit Price Contract

This type of contract incorporates a Schedule of Quantities in which all of the items of work are listed and priced on a unit price basis, to give a total contract price. This type is primarily used on sewer, watermain and roadwork projects and on excavation jobs where it is difficult to state the exact volume of earth and rock to be encountered. In this contract the Contractor is paid for whatever work he carried out and one can see that the final value of the contract can fluctuate significantly, depending on how accurately the original quantities were estimated.

The role of the Inspector in this type of contract is to ensure that all the work completed by the Contractor is properly measured and documented and that the work has been carried out in accordance with the drawings and specifications.

Cost Plus Contracts - General

In this type of contract, the Contractor is paid the cost of his materials, equipment, labour and overhead, plus a profit. This is a very difficult type of contract to administer and requires an on-site bookkeeper as well as inspector. Under cost plus arrangements it is important to agree, beforehand, what costs will be considered as relevant. There are three types of cost plus contracts namely:

Cost Plus Percentage Contract

In a cost plus percentage contract, the Contractor is paid the actual cost of work, plus an agreed percentage of the actual or allowable cost to cover overhead, profit, etc. It is useful in an emergency, when there is insufficient time available to prepare a detailed scheme or when the work cannot be clearly identified because of uncertain conditions.

The major disadvantage of this form of contract is the lack of incentive for the Contractor to complete the work as quickly as possible or to try to reduce costs. While there is some assurance to the Owner that there are no extravagant profits being paid to the Contractor, it is necessary to closely monitor and control the Contractor's records relating to the cost of the work to ensure that proper accounting is carried out.

Cost Plus Fixed Sum Contract

In this form of contract, the Contractor is paid the actual cost of the work plus a fixed sum, which has been previously agreed upon and which does not fluctuate with the cost of the job. It is superior to the cost plus percentage contract, as there is some incentive to the Contractor to complete the work as speedily as possible, although once again, the work records should be carefully checked and monitored.

Cost Plus Fixed Sum, with Bonus or Penalty, Contract

In this type of contract, the Contractor is paid the actual cost of the work with a fixed sum previously agreed upon. In this case, however, if the Contractor is able to reduce the cost of the work below the original estimated costs, he receives an agreed percentage of the savings, while if he allows the cost to exceed the estimate, he is penalized by deduction from the agreed fixed sum, a percentage of the excess of the actual amount over the estimated cost. As this form of contract incorporates an incentive to carry out the work as quickly and cheaply as possible, it constitutes the best of the "cost plus" type of contract, from the Owner's point of view.

CONTRACT DOCUMENTS

Every written contract must define exactly the rights and obligations of the parties involved, as well as provide an explicit description of the work to be completed. In addition, as the offer and the acceptance are part of the contract - since it is through them that the contract is created - there should also be a formal contract agreement, particularly in large complex projects, which involve several interested parties.

Accordingly, the usual construction contracts for typical Civil Engineering projects include the following basic elements:

Agreement
General Conditions of Contract
Specifications
Special Conditions
Contract Drawings
Tender

On the more complex contracts, involving a number of different facets, one may often encounter in addition:

Information to Tenders
Addenda, if any
Supplementary General Conditions
Supplementary Specifications

Each element provides a specific contribution to the mutual understanding and makes the contract a clear, precise and legally binding document.

Agreement

The Agreement is a document which formalizes the contract by bringing together all the other contract documents by reference to them. The purpose of the Agreement is to record the business and financial arrangements between the Owner and the Contractor with respect to the work. This is the most formal part of the contract documents and should be entirely devoid of technological material making it unnecessary for field staff to have access to it.

In the Agreement, the Contractor covenants to construct, complete and maintain the works in accordance with the contract, and the Owner covenants to pay the Contractor at the times and in the manner prescribed by the contract. This is the document that is usually signed, sealed and witnessed by both parties.

The agreement also sets out the order of precedence of the various contract elements in the event that there is a conflict between clauses in two or more of these elements. The problem of conflicts is discussed in more detail later in this topic.

General Conditions of Contract

The General Conditions define the legal rights and obligations of the Owner, the Contractor and the Engineer, the Owner's agent, and may be described as the regulations under which the contract will be performed and as such, set the pattern for administration of the contract.

Although the General Conditions cover a number of topics of a work project, the Owner will arrange and adapt the topics he finds necessary to suit his own unique requirements.

Typically, the topics covered in the General Conditions fall in one of the groups outlined below.

Group	Topics
Scope of Work	Definitions, estimated quantities, changes and alterations, extra work, acceptance.
Control of Work	Engineer's authority, claims and negotiations, conflicts and omissions, line and grade, damage by vehicles.
Control of Materials	Supply of materials, source of supply, defective materials.
Legal Responsibility	Contractor's responsibility for damages, liability insurance, access to properties, explosives, observance of laws, statutes and regulations, notice to Contractor notice to Owner.
Prosecution and Progress	Time and order of completion, extension of time, assignment and subletting, non-fulfilment of contract, suspension of work, losses and damages, labour disputes, obstructions, limitation of operations.
Payment	Price for work, advance payments, payment for extra work, measurement of materials, Contractor's discharge of liabilities, payment of workmen, books, payrolls, accounts and records.

Specifications

This document sets out the technical requirements of the work. It describes in detail the work to be executed under the contract and the nature and quality of the materials and workmanship. As well, it acts as legal evidence in the event of litigation.

For each phase of the work, there is a specification prepared to describe in clear, concise and simple language, exactly what is required. While there are various forms in which the Specifications may be written, it is essential that certain information is brought out clearly. The following headings indicate the key points to be included.

1. Description

   Provides a full explanation of the job task and of what is expected of the Contractor in carrying out his work.

2. Materials

   Identifies the technical requirements of all the materials necessary to complete the work.

3. Construction

   Advises the Contractor of the final product to be provided and of the general construction practices to be followed. Where the contractor feels that he has a more efficient or economical approach he would contact the Engineer for approval to use his alternative method.

4. Measurement

   Outlines the method of measurement to be used in determining the quantity for which the Contractor will be paid at the contract unit price.

5. Payment

Describes the basis of payment, specifying precisely what the Contractor is required to do for the contract unit price of the particular item.

Special Conditions

This document is in reality a supplement to the General Conditions discussed earlier, and it identifies the unique peculiarities of the specific project. Consequently the Special Conditions are prepared separately for each project and deal with matters such as duration of construction, work program scheduling, damages payable for late completion, maintenance period after completion, unusual administrative arrangements, special constraints placed on the Contractor and noteworthy site conditions.

Contract Drawings

These are the pictorial representations of the project, showing clearly, simply and completely the extent of the works to be carried out under the contract. The drawings should also indicate all pertinent existing information such as contours, man-made and natural surface features, and sub-surface conditions where applicable. It is essential that the drawings be accurate, with legible and adequate dimensions to properly convey the required information to the Contractor. It has been proven that the better the quality and completeness of the drawings, the less opportunity there is for disputes and misunderstandings.

Form of Tender

This document lists the various items of work included in the project along with their estimated quantities. Often otherwise referred to as the schedule of quantities, schedule of unit prices or schedule of rates, this document provides a basis on which Contractors can prepare bids and when the various Tenders are received, affords a means of comparing the prices. After the contract is signed, the Tender forms the basis for valuation of variations and extra work; while during actual construction, it is valuable in the preparation of progress payment certificates.

Information of Tenderers

This section provides the would-be bidders with the information regarding the tendering procedure. The type of information provided includes:

1. availability of tender documents
2. time, date and place of tender closing
3. amount of tender deposit
4. necessary bonding and certifications
5. additional required information about the Contractor
6. project description
7. special tendering requirements
8. interpretation and discrepancies

Addenda

Changes made to the project after advertising but prior to Tender close are designated by means of addenda, issued to all plan-takers and are considered as part of the documents. As part of the Contractor's submission of his Tender, receipt of the addenda must be acknowledged and the requirements stated therein included in the submitted Tender price.

Supplementary General Conditions
Supplementary Specifications

Under special circumstances, it may be necessary to include Supplementary General conditions or Supplementary Specifications into the contract documents. Although these items may alternatively be included under Special Conditions, setting them out separately emphasizes them - a useful alternative in cases where site conditions or project administration dictate the needs for the modifications to the clause in the General Conditions.

CONFLICTS

All of the elements discussed above and as set out by the Consulting Engineer, constitute the

contract documents and as such they should be in agreement amongst themselves. However, as sometimes occurs, there is a conflict between two or more of the elements and the courts historically have indicated that all of the documents are not of equal status.

The most common contractual document used in Canada for almost all private arrangements is the Canadian Standard Construction Document, which is available for various forms of contract including the lump sum price contract and the Unit Prices contract.

This document outlines the priorities in the event of conflicts between contract document as being:

1. Documents of a later date shall govern.

2. Figured dimensions shown of the drawings shall govern even though they may differ from scaled dimensions.

3. Drawings of a larger scale shall govern over those of smaller scale of the same data.

4. Specifications shall govern over drawings.

5. General Conditions of a contract shall govern over Specifications.

6. Supplementary General Conditions shall govern over the General Conditions of the Contract.

7. The Agreement shall govern over all Documents.

In the event of an Addendum being issued, it shall supersede the Supplementary General Conditions, but not the Agreement.

Although this is a common and well-known approach to the question of conflicts between contract documents, other agencies such as the Ontario Ministry of the Environment and Ministry of Transportation and Communications have developed separate formats which are only slightly different but suit better the particular Ministry's purposes.

In any case, whatever the order of precedence may be for a contract, it should nevertheless be clearly set out in the Documents to ensure that such disputes can be quickly resolved.

DISCHARGE OF CONTRACTS

A contract is said to be discharged when the agreement between the parties is terminated, and the rights and liabilities arising out of the contract are as a result extinguished.

This termination may be accomplished in several ways.

1.	By Agreement	The parties may agree to terminate the contract by one either waiving his rights, or releasing the other from his obligations.
2.	By Performance	Complete fulfilment of the terms of the contract in every respect, by both parties, discharges the agreement entirely.
3.	By Breach	Whenever one party breaks an obligation which the contract imposed upon him, he has breached the contract. Depending upon the circumstances, the other party may discharge the contract.
4.	By Impossibility	Legal impossibility or impossibility of performance may arise after the contract by law from the date when the cause of action arose and if the action is not commenced by that date, the contract is considered void.

CONCLUSION

This brief discussion on the basic elements of contract documents provides only a simplified summary of the various aspects of contract preparation. Specific points should be discussed with legal counsel, if there is some concern about inherent implications and responsibilities related thereto.

SUBJECT:

CONSTRUCTION OF SEWERS WATERMAINS AND ASSOCIATED FACILITIES

TOPIC: 5

ENVIRONMENTAL CONSIDERATIONS

OBJECTIVES:

The trainee will be able to:

1. Better understand the Environmental Assessment Act, 1975.
2. Recall why certain environmental control procedures are followed during construction.
3. Describe specific procedures for environmental control.

ENVIRONMENTAL CONSIDERATIONS

GENERAL

Throughout the 1970's and early 1980's, the public began taking a greater interest in the effects of large-scale projects such as roads, sewers, sewage treatment plants, pipelines, river diversions, nuclear power plants new towns and international airports. In many instances the public objected to projects on grounds such as personal disruption, economic disadvantages or infringement of property rights. Environmental concern was never much more than an underlying or latent issue in these objections.

Beginning in the mid 1980's, this focus changed as environmental issues assumed greater importance. Currently, there is growing public support for the belief that environmental considerations are an essential component of all stages of project planning, implementation and operation, and decommissioning.

ENVIRONMENTAL ASSESSMENT ACT, 1975

The Environmental Assessment Act (EA Act) was passed in 1975. It was made applicable to government ministries and agencies in 1976, and to municipalities in 1980. Private sector projects can be designated by the Government on a case-by-case basis.

Purpose

The EA Act is intended to provide for "the protection, conservation and wise management of the environment" through sound planning and informed decision-making.

What this legislation does is require those proposing a project (the proponent), to submit an Environmental Assessment of the undertaking to the Minister of the Environment. No other approvals can be given by any provincial or municipal agency until the Minister has accepted and approved the EA document. The emphasis has switched from the abatement approach of the Environmental Protection and The Ontario Water Resources Acts to one of avoiding impacts through comprehensive planning.

Systematic evaluation of alternatives and net effects establishes that the project is a preferred solution to a problem and ensures minimal environmental degradation. While mitigation techniques are still likely to be required, avoidance of environmental impacts is the aim. Reduced costs to the public and to the environment are the benefits of this "look before you leap" approach.

Who is subject to the Act?

The environmental assessment process requires the proponent to first determine whether the particular project falls within the scope of the EA Act. This is done through consultation with the Ministry of the Environment. Designation for government ministries and agencies, and municipalities projects is defined in the regulations issued under the EA Act (Ontario Regulation 334).

Private sector projects are not subject to the Act unless they are specifically designated by the Provincial Government. For example, landfill sites of a particular type and size are designated as subject to the Act whether the proponent is a public or private body.

Not true

ENVIRONMENTAL ASSESSMENT TYPES

There are two types of environmental assessments which can be prepared: individual and class.

An approved Class EA provides a series of planning requirements, usually streamlined and standardized, for use solely with that particular group (class) of projects. Any undertaking falling within the class has an approval under the Act if the approved environmental planning process described in the Class EA is followed. An undertaking can be "bumped up" to an individual EA for such reasons as discovery of unanticipated environmental effects or significant public controversy.

A class EA undertaking may be either a single project or several distinct activities that may or may not be similar in nature. For example, two separate but associated highways dealt with as one proposal could be similar projects within a single undertaking. As well, a flood control dam, downstream channelization, and bank stabilization works would represent dissimilar projects within a single undertaking. Include a flow chart

An individual environmental assessment includes:

- a description of the alternatives that were considered;
- their net environmental effects;
- advantages and disadvantages;
- an explanation of why the project was chosen;
- detailed description of the project for which approval is requested;
- net effects of the project;
- project design;
- site specific location for the project;
- preliminary construction schedule.

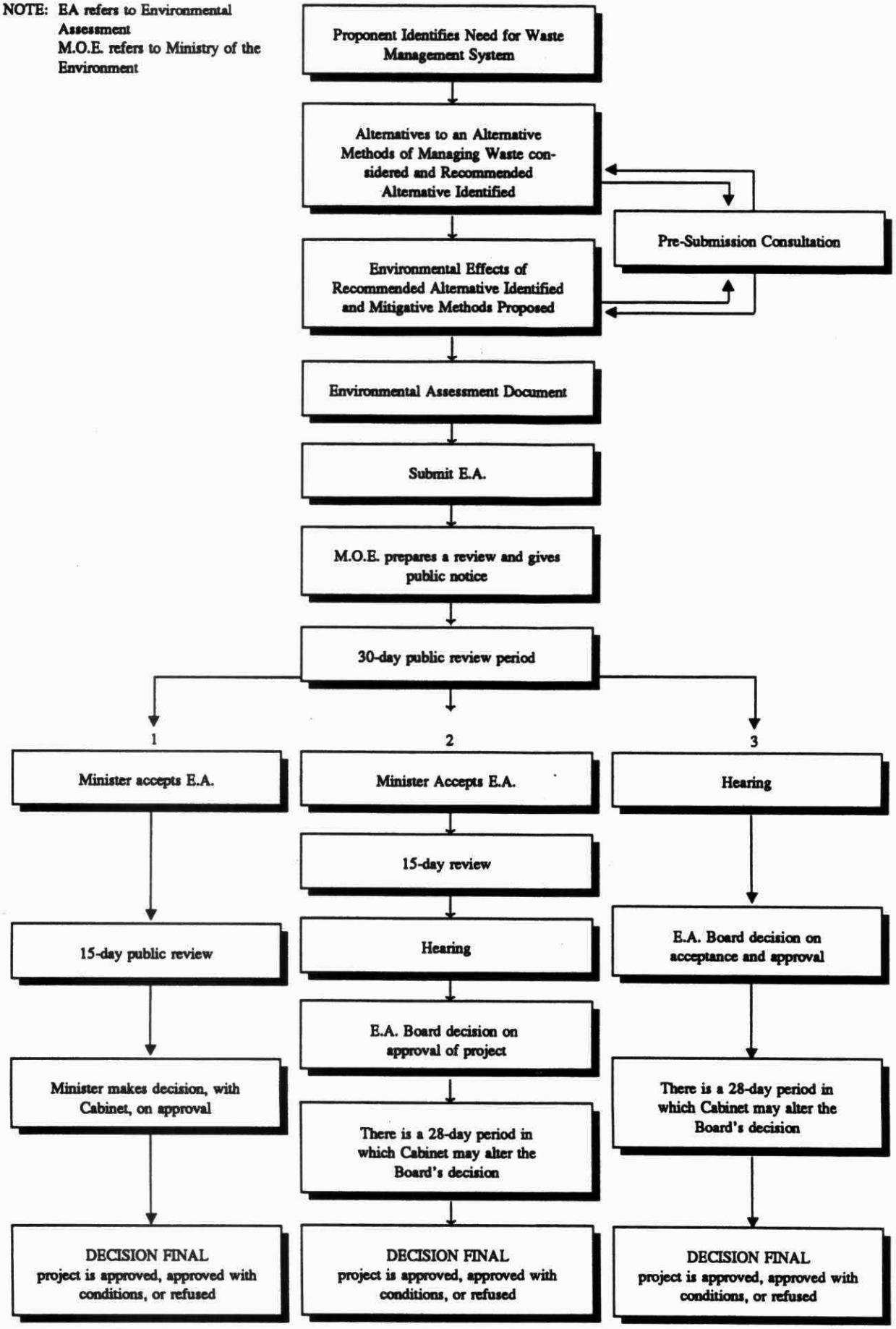
On submission, the EA document is circulated for comments to all government agencies considered to have an official interest in the project. A Review is then prepared by the Ministry of the Environment's Environmental Assessment (EA) Branch outlining the recommendations of the Review team on the acceptability and approval of the document.

The public is then given an opportunity to respond to the Environmental Assessment and the Government Review, and to request a public hearing. If a hearing is not required, the Minister of the Environment, with the approval of Cabinet, decides on the acceptability of the document and approval of the project.

The figure on page 5-4 illustrates the steps in the EA process. It shows the pre-submission consultation phase as well as the three routes a final submission can take before a decision is made.

FIGURE 1

NOTE: EA refers to Environmental Assessment
M.O.E. refers to Ministry of the Environment


Proponent Identifies Need for Waste Management System
Alternatives to an Alternative Methods of Managing Waste considered and Recommended Alternative Identified
Pre-Submission Consultation
Environmental Effects of Recommended Alternative Identified and Mitigative Methods Proposed
Environmental Assessment Document
Submit E.A.
M.O.E. prepares a review and gives public notice
30-day public review period
1
2
3
Minister accepts E.A.
Minister Accepts E.A.
Hearing
15-day review
15-day public review
Hearing
E.A. Board decision on acceptance and approval
E.A. Board decision on approval of project
Minister makes decision, with Cabinet, on approval
There is a 28-day period in which Cabinet may alter the Board's decision
There is a 28-day period in which Cabinet may alter the Board's decision
DECISION FINAL project is approved, approved with conditions, or refused
DECISION FINAL project is approved, approved with conditions, or refused
DECISION FINAL project is approved, approved with conditions, or refused

A legal requirement of the approval to proceed issued by the Minister or the Environmental Assessment Board is that the proponent is bound to carry out his mitigation techniques in accordance with what has been outlined in the environmental assessment document and the conditions of approval. This means that these techniques must be outlined in the tender documents and the plan and profile drawings by the engineering consultants so that every contractor bidding on the work is aware it represents part of the bid package. In this way there is no confusion about work to be done and the contractor cannot later submit claims for additional payment for certain mitigation measures carried out in the course of the contract. More importantly, it ensures that all of the environmental protection developed during the planning stage gets implemented in the field.

ENVIRONMENTAL CONTROL PROCEDURES

The site inspector's role is to ensure that the methods for mitigating or avoiding the environmental impacts identified in the Contract Documents are put into practice.

Mitigation practice need not be complicated, more often than not it is common sense. The following is designed to enlighten the inspector as to the measures of mitigation.

Construction Equipment Maintenance

The inspector should ~~that~~ precautions are taken to prevent the spillage of oils, fuels and other waste solutions generated during equipment maintenance. He has to familiarize himself with the procedures for the rapid clean-up of any spillage as well as the satisfactory disposal of the spilled material.

Refuelling and maintenance of equipment cannot be allowed in or adjacent to watercourses but must be performed at remote storage areas. If properly prepared, suitable refuelling and maintenance areas will have been defined at the planning stage. If not, the inspector should see that they are established.

Slippage Areas

Normally, pipe alignments are selected to give the foot of steep valley walls, which are prone to slippage, as wide a clearance as possible. In such critical areas, detailed construction procedures will have to be followed.

These would include what areas require stabilization and what material is considered suitable for that application. It is the inspector's job to make sure that those slopes which are steep and prone to slippage are given as wide a berth as possible. Where problem slopes are encountered, he should ensure that the stabilization provisions stipulated in the contract are carried out immediately following construction through the unstable area.

Right-of-Way

The construction right-of-way is designed to minimize areas of disturbance. The inspector has to see that construction equipment is kept within the designated right-of-way by utilizing stake lines or fencing if necessary. Haul routes, parking areas and so forth must be restricted and only the minimum number of routes absolutely necessary should be permitted up valley walls. Not all areas of a particular route will require a right-of-way width of 25 to 30m.

In the vicinity of areas designated as critical (ie. stream crossings, mature tree stands, etc.) the right-of-way must be limited in order to reduce adverse effects. Such limitations may necessitate a change in excavation procedures near these critical areas. Once discussions have been held with representatives of facilities or areas considered as sensitive such as golf courses, hospital zones or public recreation areas, the inspector must ensure that the most suitable alignments, construction schedules, restoration procedures, access routes, easement widths, etc. are followed.

Clearing of Vegetation

Prior to any clearing of vegetation, the routing should be examined to determine if it is necessary to remove all vegetation along the right-of-way. For example, all trees need not be removed if the pruning of low branches will suffice. The inspector must see that all those trees left standing have fencing placed around the tree at least as far from the trunk as the branches extend (the dripline). Vegetation should not be removed prematurely from the route but should only be cleared in advance of the immediate construction phase.

Restoration of vegetation, including interim measures, cannot be left as a final operation to be undertaken upon completion of the construction of the whole line, but done in stages as soon as possible after clearing. Trees that are removed are usually replaced on a one-for-one basis where it is feasible. Prior consultation will have predetermined which trees should be marked for removal and which trees should remain. The inspector must make sure these instructions on the contract drawings are followed to the letter. identify C of A?

Stockpiling of Excavated Material

The inspector must familiarize himself with the procedures for stockpiling top soil, excess excavated material, bouldery remnants, etc. Methods presently in use include interim stabilization and covering of excavated material with plastic sheeting in critical areas. Depending upon the length of time a stockpile is to remain and upon the season, the excavated material might require a perimeter ditch as well as a settling area. As a general rule, stockpiling should not be located adjacent to watercourses.

At the completion of construction, the inspector must see that any excess material is removed and placed in a more appropriate predetermined location - this does not include flood plain areas. Alternatively, the site engineer may wish to establish better contour and grades in particular areas to help restore natural site drainage.

Sediment Control and Erosion

Prior to construction, procedures are established for the minimization of erosion and control of any erosion which might occur during construction. The inspector must see that erosion is minimized by strict adherence to recommendations made in the slippage, clearing of vegetation, stockpiling, stream crossing and restoration sections of the Contract Documents.

Silt traps and ditching devices along the construction route, as indicated in the plans, are designed to help control erosion that occurs (especially during storm periods). The inspector should ensure that any control device called for in the specifications is installed prior to construction activities. He will determine if the silt traps need to be re-excavated (and in such a manner which will not introduce sediments to the watercourses) in order to maintain their efficiency.

Placement of Soil in Woodlots

Frequently soil material excavated from a right-of-way is placed on either or both sides of it. The access to woodlots along rights-of-way should be restricted by the use of snow fence and intensive educational programs. If trees are to be saved, soil should not be piled around root systems nor should heavy equipment be allowed to pass close to the trunk. Also, overhanging branches should be removed prior to the movement of mechanized vehicles along the right-of-way. Where damage does occur, surgeon paint should be applied.

Stream Crossing

Stream crossing and the amount of stream bank modification should be kept to a minimum. Necessary crossings should be at right-angles or as close to right angles as possible. The inspector must ensure that the stream crossings are conducted according to the contract specifications. Such features as diversion culverts and fill material must be suitable for that application and properly positioned. Regarding the crossing itself, the clearcut right-of-way of the approach should be limited. The number of points that heavy equipment can cross the river should be restricted and at no time should the stream bed be used as a route. Backfill must be satisfactory to prevent erosion and washout. The actual crossing should be planned to coincide with low flow periods and the work staged to minimize the time of soil exposure.

Discharge to a Watercourse

Any water that is collected or removed from trenches in the construction zone must not be discharged directly into a watercourse until adequate treatment has been provided to remove suspended silt, clay and other contaminants. If treatment prior to discharge into a watercourse is not considered practical, consideration should be given to another means of disposal such as discharging on grassy areas sufficiently remote from the site. Where the water is carrying heavy silt loads it should be directed to sedimentation ponds. Constructed on site

Groundwater

On certain projects, the inspector may have to deal with a shallow perched water table. Where a pipe, especially a sewer, intersects a shallow perched water table, extra precautions are necessary to ensure that the sewer is impermeable in order to minimize the possible effects of leakage and groundwater contamination. Water pumped out for purposes of trench de-watering should not be discharged directly to watercourses but handled as outlined above. Also, the inspector must ensure that there is minimum disturbance of areas of groundwater seepage, particularly near steep slopes.

Dust Control

Associated with most construction projects is the problem of dust control. Depending upon the time of the year and the weather conditions, conditions may become quite an annoyance. The inspector should exercise his judgement in determining whether some control measure is necessary and what method might be most appropriate in a particular situation. A general rule of thumb to follow is that if the construction is in an urban area, particularly in the summer, periodic spraying with water or the application of calcium chloride is good preventative medicine.

Hydrostatic Testing

Water used in the hydrostatic testing of the pipeline should not be discharged directly to watercourses. It is recommended that the discharge be buffered with a holding lagoon or alternatively a discharge pipe and diffuser properly positioned in relation to the receiving waters.

These decisions will have been made prior to the testing so that the inspector can enforce the specified method. The rate of discharge to and from the holding lagoon to the watercourse must be regulated in order to preclude erosion and sedimentation in the watercourse. Water containing suspended silt, clay and other contaminants should not be discharged to the watercourse without adequate treatment.

Restoration

One of the most important aspects of any construction project is the site restoration. Once the pipe is in the ground, people no longer are concerned about it but they will become concerned very quickly if some feature above ground is not returned to its original state.

Prior to construction, procedures will have been defined for the interim (i.e., mulching, use of netting, seeding of grasses) and long-term rehabilitation (replacement of vegetation) of areas disrupted by the construction. As far as possible, disturbed terrain should be returned to its original contours. However, the drainage in certain areas can perhaps be improved upon where considered desirable.

Other landscaping features such as tree replanting must be carried out in accordance with the specifications and the inspector is to see that the work is conducted satisfactorily. Follow-up procedures can assess the effectiveness of the restoration and improve any areas of deterioration and inefficient restoration. For example, additional work may be required to remove the effects of settling.

There and the many related aspects of site rehabilitation are discussed at some length of the "Restoration" section of this manual.

Conclusions

If "ad hoc" environmental controls are avoided, then the planning, design and implementation of a project can proceed with controlled environmental effects. The contractor and inspector will be ready at each stage to implement various mitigation and control measures.

The inspector has the ultimate task to ensure terms and conditions are met and, to ensure the contract is fulfilled in an environmentally effective (cost, time, and environmental protection) manner.

SUBJECT:

CONSTRUCTION OF SEWERS
WATERMAINS AND ASSOCIATED
FACILITIES

TOPIC: 6

CONSTRUCTION SAFETY

OBJECTIVES:

The trainee will be able to:

1. Interpret and implement the Occupational Health and Safety Act and the Regulations for Construction Projects, noting differences between the duties of the Owner's inspector and those of the Ministry of Labour inspector.

2. Become more familiar with the sections of the Regulations which affect trenching.

3. Give some direction to the actions which should be taken to ensure the safety of personnel on the project.

CONSTRUCTION SAFETY

INTRODUCTION

Every year in Ontario about 35 workers are killed and thousands of others receive injuries ranging from minor cuts and bruises to total disability. The enormous financial costs involved are overshadowed by the human costs to workers and their families after serious accidents.

The major causes of death and disability in construction are:

1. Falls between different levels.
2. Electrical Contact.
3. Struck by vehicles and equipment.
4. Cave-ins.
5. Confined space accidents.

All of these types of accidents can occur on a sewer and water project and their prevention, regardless of legal obligation or apparent degree of responsibility, is everybody's business.

THE LAW

Safety in the Ontario workplace is governed by the Occupational Health and Safety Act and various Regulations which cover different workplaces - e.g. Construction Projects, Industrial Establishments, Mines and Mining Plants etc., or different substances encountered in the workplace - e.g. Asbestos, Lead, Silica, Hazardous Materials (WHMIS) etc. This legislation is administered and enforced by the Ministry of Labour.

CONTRACT DOCUMENTS

In the specifications there should be clauses indicating that the contractor must comply with the requirements of the Occupational Health and Safety Act which are the minimum requirements for health and safety on a project. This means that the Act, and the Regulations under the Act, form a part of the contract for the work to be done.

There should be clauses similar to the following indicating the inspector's authority under the contract:

Inspector and Inspection

In particular, but without limiting the powers of the Inspector, orders given by the Owner's Inspector relating to the quality of material or workmanship or in respect of safety or public convenience must be obeyed at once by the Contractor. The Owner's Inspector shall have the power to require the Contractor to suspend workers for incompetency, negligence, impairment or disregard of orders, and the Contractor shall ensure that any worker so suspended is forthwith removed from the site.

Public Convenience and Safety

If at any time the Engineer or his authorized representative consider the works to be unsafe, he may order the Contractor to take measures forthwith to ensure safety. Should the Contractor fail to take adequate measures, the Engineer or his representative may order the work to cease until such measures have been taken. The Contractor shall not be entitled to additional payment for, or an extension of time for the performance of the contract by reason of such safety measures. The fact that the Engineer or his representative has ordered or has failed to order additional safety measures shall not relieve the Contractor of the responsibility for the adequacy of the safety measures taken.

The minimum acceptable quality of safety on a project is contained in the Occupational Health and Safety Act and the Regulations for Construction Projects 213/91. The law requires that the contractor post a copy of the Act on the project at all times. A copy of the Act and Regulations should also be available in the Owner's inspector's office.

The Occupational Health and Safety Act (Chapter 321 amended)

The Act covers almost all workers in Ontario and defines the rights and responsibilities of all workplace parties.

The Act is administered and enforced by inspectors reporting to a Director of the Occupational Health and Safety Division of the Ministry of Labour. The inspectors make periodic visits to construction projects in their area and have the responsibility to deal directly with the contractor. They leave written directives with the contractor for any infractions which have to be corrected. These directives must be acted upon immediately. The inspectors also investigate workplace fatalities, critical injuries and work refusals.

Section 1 of the Act, defines terminology used in the legislation. The Owner's inspector should pay particular attention to some of these definitions as they clarify certain sections of the Act and Regulations. Some examples are competent person, constructor, employer, owner, project, supervisor, work place and worker.

Please note that the numbering of the sections of the Act in this document correspond to the Ministry of Labour's renumbering effective January 1, 1992. Therefore, the section numbers in this document may not correspond to those in your current copy of the "green book".

The duties of the various parties are described in the following sections of the Act:

Constructor	Section 23
Employer	Section 25 and 26
Supervisor	Section 27
Worker	Section 28
Owner	Section 30
Supplier	Section 31

On reading through the Act the Owner's inspector may not feel responsible for enforcement of the legislation. However, the inspector is a worker and therefore has a duty to report hazards and contraventions of the Act and Regulations to his or her supervisor (sect. 28).

Familiarity with the legislation will allow the Owner's inspector to point out to the contractor any infractions which should be corrected. These should be given in writing to the contractor and noted in the job diary. If the contractor neglects to take appropriate action, and safety provisions are being contravened or ignored, the Owner's inspector should notify his or her supervisor then contact the Ministry of Labour and request that a Ministry inspector visit the site.

The Owner's inspector should also be familiar the right to refuse work where health and safety are in danger (sect. 43), and also the penalties that can be incurred through contravention of the Act (sect. 66). Contravention of the Act or Regulations can result in a fine for an individual of up to $25,000 and/or a one year jail term. The maximum fine for a corporation is $500,000.

Ontario Regulation 213/91 (Construction Projects)

The Regulation is broken down into the following parts:

Part I	General (Sections 1-19)
Part II	General Construction (Sections 20-221)
Part III	Excavations (Sections 222-242)
Part IV	Tunnels, Shafts, Caissons & Cofferdams (Sections 242-331)
Part V	Work in Compressed Air (Sections 332-399)

Parts I and II apply to all projects but the majority of the sections of concern to the sewer and watermain construction inspector are found in Parts II and III. The following information highlights sections of the Regulation that are of particular importance on a sewer and watermain project. However, the inspector should have some knowledge of the entire Regulation. A review of the Table of Contents and Index would be a useful exercise.

Part I General (Sections 1-19)

Section 1 defines terms used throughout the Regulation and should be fully understood eg. Confined Space, Trench, Shaft etc.

Sections 5-12 deal with registration of employers and employees and with notices required by the Ministry of Labour. The inspector should be familiar with these sections so that the proper information can be reported to the supervisor in the event of an accident or incident.

Part II General Construction (Sections 20-221)

Sections 21-27 Protective Clothing and Equipment: Hard hats and safety footwear must be worn at all times on a project. Other equipment must be used as circumstances dictate.

Section 28-30 Hygiene: Requirements for drinking water, toilet and clean-up facilities. Compliance can be a problem on small or short term projects.

Sections 35-43 Housekeeping: Waste material must be removed to a storage area daily. Materials to be moved by crane must not be stored under or close to overhead electrical wires. Materials must not be stored closer than 1.8 metres from the edge of an excavation.

Sections 52-58 Fire Safety: This section deals primarily with fire safety in buildings under construction. However, it is applicable to sewer and water construction if flammable materials are stored on site or if welding or cutting is taking place.

Section 59 Dust Control: Dust from rock drilling must be collected or controlled. If this is not practicable, adequate respiratory equipment shall be provided and used by workers.

Sections 60-63 Confined Spaces: It is essential that the inspector be familiar with the requirements for atmospheric testing and rescue before the inspector or any worker tries to enter a manhole, valve chamber, tank, wet well, dry well or any other confined space.

Sections 64-66 Public Way Protection: The public must be protected by covered ways, fencing and hazard marking.

Sections 67-69 Traffic Control: Workers must be protected from traffic by traffic control persons, warning signs and lane control devices etc. All workers on the public right of way must wear fluorescent safety vests.

Sections 70-74 Access, Egress, Ramps etc.: Proper access to all work areas, including trenches, must be provided.

Sections 78-84 Ladders: The inspector should be aware of the proper construction, use and maintenance of ladders.

Sections 93-116 Equipment: The inspector should pay particular attention to sections 104 to 106 which deal with the requirements for a signaller when the operator's view is obstructed.

Sections 181-195 Electrical Hazards: Particular attention should be paid to section 186 regarding the safe approach distances to live, overhead lines. Disregard of the limits of approach is a major cause of fatality in the trenching industry.

Sections 196-206 Explosives: These sections deal with the competency of the worker in charge of handling, transporting, placing and firing explosives. Local bylaws may apply when explosives are used. Also, the Explosives Act of Canada and sections 79 to 82 of the Criminal Code of Canada apply.

Part III Excavations (Sections 222-242)

Sections 222-223 Interpretation & Application: Defines engineered support system etc.

Sections 224-225 Entry & Working Alone: While work is proceeding in a trench there must be a worker at ground level close to the trench.

Sections 226-227 Soil Types: Defines soil types 1,2,3 and 4 and describes how soil should be classified.

Sections 228 Precautions Concerning Services: All gas, electrical and other services must be accurately located before excavation starts. These services must be adequately supported during construction.

Section 229 Protection of Adjacent Structures: Precautions to protect adjacent structures must be specified by a professional engineer.

Sections 230-233 General Requirements: These sections deal with control of water in the excavation and other factors affecting stability. Section 233 contains a footnote referring to the Criminal Code with respect to the duty of a person in charge of an excavation.

Section 234-242 Support Systems: Section 234 describes when support systems are not required eg. in Type 1 or Type 2 soil when the walls are sloped to within 1.2 metres of the trench bottom at a minimum slope of one to one. The remaining sections detail the requirements for timber shoring, prefabricated, hydraulic or engineered support systems. The inspector should note that prefabricated or hydraulic support systems are not permitted in Type 4 soil conditions.

Part IV Tunnels, Shafts, Caissons & Cofferdams (Sections 243-331)

The requirements of Part IV will not be encountered often. However, the inspector may be involved in short tunnels such as those under roadways and railways. It is just as important that the Regulation be adhered to in such situations and the inspector should make sure that short cuts are not taken.

The inspector should pay particular attention to sections 248 to 259, fire protection, and to sections 264 to 268, rescue of workers.

Part V Work in Compressed Air (Sections 332-397)

Work in compressed air is highly specialized and beyond the scope of this course. If inspection in a compressed air environment is anticipated, special training should be carried out, including a detailed review of this Part of the Regulation.

GENERAL

Some of the basic tools an inspector needs on the job site are a sharp eye, an enquiring mind and a civil tongue. Spotting potential problems and communicating them to the contractor diplomatically can save much of the grief brought on by the adversarial system.

The inspector can work up a routine for checking on safety as he walks the site:

1. Are barricades, delineators, signs, flashers etc. in place and in working condition?

2. Are all workers wearing the personal protective equipment required for the job they are doing?

3. Are the sides of the trench clear for walking? Is the backhoe casting well clear of the trench? Are pipes and equipment clear of the edge of the trench?

4. Are ladders in place for easy access and egress?

5. Is the backhoe operator keeping the boom well clear of overhead wires?

6. Are the sides of the trench stable?

7. Are there any cracks in the ground surface near the trench indicating possible movement of the ground or shoring?

8. If workers are required to enter a confined space such as a manhole, are they properly trained and equipped? Do they have adequate means of rescue?

Items such as these can be checked very quickly. Deviations from normal practice may indicate that the contractor has a problem or is becoming careless with regard to safety.

Remember, every worker has the right to a safe, healthy workplace. You can contribute to safety in the workplace by making the contractor aware of potential hazards and violations of the Occupational Health and Safety Act and the Regulations for Construction Projects.

SUBJECT:

CONSTRUCTION OF SEWERS
WATERMAINS

TOPIC: 7

GUIDELINES FOR TRAFFIC
CONTROL IN A
CONSTRUCTION AREA

OBJECTIVES:

The trainee will be able to:

1. Understand the existing guidelines and regulations governing the Control of Traffic in Construction areas with particular emphasis on the safety aspect.

2. Provide direction to any action required, if not already taken, for safe control of traffic in a Construction area.

GUIDELINES FOR TRAFFIC CONTROL IN A CONSTRUCTION AREA

GENERAL

Guidelines exist to provide for uniform control of traffic on public road allowances and where special conditions are required these are usually specified in a supplementary form. All guidelines are subject, of course, to specific site conditions and they should complement but not substitute for traffic engineering expertise.

While the responsibility of implementing the standard and special conditions for traffic control falls on the Contractor, the traffic engineer (or co-ordinator) and the police, the construction inspector by virtue of his presence at the site most if not all of the time and his knowledge of the progress of the construction work and scheduling, has a part to play to ensure that the required conditions are put into effect. This is to allow not only for the maintenance of some vehicular traffic (though likely to be restricted) where permissible, through the construction area, but also give high priority to pedestrian traffic and foremost to consider safety. This includes safety to the pedestrians in the immediate area, safety to the construction workers and safety to the vehicular traffic permitted in the area.

In dealing with construction in an urban area, special consideration must be given to the maintenance and safety of pedestrian traffic with particular attention to the aged, the young and the handicapped. This is especially the case where construction interrupts a route to a shopping area or a school or a recreational facility (park, swimming pool, skating rink, tennis courts etc.)

REFERENCES

1. Ministry of Transportation and Communications - Traffic Control Manual for Roadway Work Operations.

2. Occupational Health and Safety Act, Ontario Regulation 213/91.

3. Local Municipality, Regional Municipality or Metropolitan Municipality - Guide for Control of Traffic during construction in the Municipality.

4. The Construction Safety Association of Ontario - Construction Traffic Controller's Handbook.

5. Province of Ontario - Highway Traffic Act.

6. Province of Ontario - Highway Improvement Act.

7. Local Municipal Traffic By-Laws.

8. Contract Documents - for Standard Requirements.

PRE-CONSTRUCTION PLANNING

The Construction Inspector should be aware that before a tender is called for utility and/or road construction or reconstruction, the design engineers and traffic engineers would have to consider the effects of construction on traffic and the special requirements for that particular project. Some of the consideration include:

1. Construction Scheduling and Restrictions if any

   There may be a case where the Contractor cannot be permitted to work unrestricted and a schedule or restrictions have to be set for him to follow, example: no lane closures during rush hours. It may be the case that we cannot have construction on a certain street or intersection at a certain period, example: to accommodate certain events - parades, sports events, agricultural fairs, etc. There may be a preference for a Contractor to work on certain areas or a street at a certain time, example: on a school route, therefore, the contractor is to work during the summer school vacation.

2. Permissible Road Closures, Detour Roads, Maintenance of Vehicular Traffic, Maintenance of Pedestrian Traffic

   From the construction point of view is ideal for construction to proceed in an area where there is little or no interference from vehicular and pedestrian traffic other than that of the operations. In an urban area, conditions are generally less than ideal in that access has to be provided to local residences, to industries and commercial establishments and for emergency vehicles. At best the road where construction is to be carried out should be closed to through traffic but allow local traffic only. This is the case provided that there are convenient routes for detour purposes. Where there are no convenient detour routes, and should the permissible traffic (both local and/or through type) be much of a hindrance to the construction operations, consideration is then given to the construction of a detour road alongside the construction area, provided space is available.

   Consideration will also have to be given to the types of industries or commercial establishments in the area and whether provisions have to be provided for normal or special deliveries re - frequency and heavy loading.

The maintenance of pedestrian traffic through a construction area and the safety of the pedestrians can never be over emphasized.

Where there is heavy pedestrian traffic then it is considered and specified that set routes be maintained or constructed for pedestrian movements with the necessary protection in the form of barricades. It may even be necessary to conduct a pedestrian tunnel.

3. Standard and Special Signing

It is highly undesirable to have approaching traffic to or traffic in a construction area confused with inadequate or improper signing. Pre-planning is carried out to list and locate the required signing to properly inform or direct motorists around or through a construction site for the various stages of construction. Early warning is also an incentive to oncoming traffic to seek alternate routes and not converge to proceed through the construction area and hinder construction.

4. Transportation Routes

The Transit Company is consulted with, and should their route be interrupted, alternative routes are planned and scheduled.

5. Authority - By-Law

The closing of a street, changing the status of a street from one way to two way or vice versa cannot be simply done by merely putting up signs. Should these changes be required, the passing of a By-Law to authorize the change may be necessary. This may vary for different municipalities and has to be checked with the municipality's legal representatives. Advanced processing is essential to have all legal aspects cleared before construction commences.

6. Notification to Others

Any authority which may carry out any construction within a road allowance in any shape or form should be notified of any proposed construction and the likely change of traffic pattern arising therefrom. While this may in addition co-ordinate some simultaneous construction in an area, it is to ensure that any diverted traffic will not run into construction hindrances planned by others. (i.e. utility companies, developers etc.).

CONSTRUCTION INSPECTOR'S ROLE IN CONTROL OF TRAFFIC IN A CONSTRUCTION AREA

The pre-tendering investigations and procedures have been completed, special conditions have been written into the contract documents, the contract has been called, opened and awarded and construction will soon commence. The construction inspector will now, though not directly, be involved in control of traffic in a construction area.

The Construction Inspector should familiarize himself/herself with the traffic requirements and signing as specified in the contract documents. He/she should attend the pre-contract or site meetings and be fully aware of the contractor's scheduling (where available) and at these meetings obtain contacts with the Police, the Traffic Co-ordinators, the Transit Commission and his/her superiors to ensure that he/she knows what exactly is required at each stage of construction. Should there be variance, he/she would report to the relevant bodies, so that changes, if required, can be made.

The Construction Inspector should also be aware of other non-scheduled construction in the vicinity which may affect traffic in a construction area for that particular municipality.

The Construction Inspector, with a knowledge of standard practices for the control of traffic on construction sites and of the special requirements for a particular project, may have certain traffic related duties in the absence of a traffic inspector, and these include:

1. Checking signs to ensure that the contractor has placed signs as required by the Traffic Engineer and/or specified in the contract documents or discussed at the pre-contract or other meetings. (Signs should not only be placed at the construction site, but also at the roads leading to or approaching the construction site).

2. Checking to ensure that the contractors have the regulation sizes or approved types of signs and that they are not some old sub-standard signs dug up from the yard and brought to the job site.

3. Checking that all delineators are properly placed lanewards and checking the spacing if there is channelization of traffic.

4. Notifying the public (residential, industrial or commercial occupants), the Police, Ambulance, Fire Department, Transit Commission etc. of any obstruction to traffic or checking to see that these notifications are carried out if this is done by others. This will also include any changes taking place in the course of construction as it is inevitable that changes will take place as contractors proceed along the job. Signs may have to be relocated because their present location is in conflict with the contractor's operations. There may be other agencies working in the vicinity or even on the same construction site. Emergency work maybe carried out to repair a broken watermain, a leaking gas main or a plugged sewer, etc.

5. Reporting to the Traffic or Construction Engineer for approval of any proposed changed.

Summing up, it is imperative that the Construction Inspector check with, report to, or consult with the Traffic Engineer/Coordinator at all times with regard to any changes or changes proposed by the contractor or by himself/herself for traffic control on construction sites. By being on the construction site most if not all the time and by watching the contractor's operations, he/she may be able to make suggestions which were not taken into consideration or did not appear to be warranted at an earlier stage.

APPENDIX 'A'

Appendix 'A' consists of extractions from the "Traffic Control Manual for Roadway Work Operations, Ministry of Transportation and Communications, Province of Ontario". Division 5 - Temporary Conditions.

These are some excerpts and reference should be made to the manual for complete section.

Part 1	General Instructions
Part 2	Sign Schedule
Part 3	Signing or Work Operations

GENERAL INSTRUCTIONS

A5.01 INTRODUCTION

A5.01.01 Purpose

The Traffic Control Manual for Roadway Work Operations has been developed to provide basic uniform requirements for traffic control in work areas during roadway or utility construction and maintenance operations on or adjacent to any public highways, municipal roads or streets and is for use by the following agencies and organizations:

1.) The Provincial and Municipal road authorities in Ontario and their contractors.
2.) Public utilities, contractors and others who may have approval to work on public roadways.

The purpose of this manual is to provide the basic **MINIMUM TYPICAL** standards for traffic control to be used by persons or agencies performing construction, maintenance and utility work on any street or highway open to the public in the Province of Ontario.

A5.01.02 Application of Standards

The standard use of traffic control devices is illustrated for typical and most common situations. Hills, curves, intersections, driveways and other physical features may require variations from the standards, and judgment must be used to select the most appropriate devices. However, all installations shall conform to the general principles and standards presented in this Manual.

With the possible exception of routine maintenance by Bell Canada, Ontario Hydro and routine municipal maintenance and utility operations as exempted by the road authority, no work should commence without first obtaining a work permit or the approval of the road authority from the road jurisdiction concerned.

It is difficult to illustrate detailed standards of applications for all the situations that may arise. Where special situations occur, additional protection should be provided. Such protection should be determined on the basis of traffic speeds, vehicle volume, sight distance, duration of operation and exposure risk to hazards. The Traffic Engineer or Road Superintendent of the road jurisdiction should be consulted before any deviation is made from this Manual and for guidance in special traffic control circumstances.

The standards outlined in this Manual shall be adopted by all public authorities concerned with safety in construction and maintenance areas on public highways and streets. Instructions should be given to all employees and contractors and the standards shall be included into specifications for all contracts.

A5.01.03 Fundamental Principles

Traffic Safety should be designed into construction and maintenance projects, rather than applied on a makeshift basis. Traffic should be routed through these areas in a manner which most closely resembles normal roadway conditions, but just as importantly, ensures the safety of the workers.

It is recommended that a traffic control plan be prepared in advance for any significant projects, and that the agreement of the road authority be obtained.

If possible, maintenance and construction activities should not be scheduled during peak hours, typically, 7:00 to 9:00 a.m. and 4:00 to 6:00 p.m.

Traffic movement should be interfered with as little as possible. Frequent or unnecessary changes such as sudden lane narrowing, lane closings, reductions in speeds and risky maneuvering shall be avoided. Special precautions shall be taken to ensure that construction equipment can be operated safely without making it hazardous to passing traffic.

Where interference is unavoidable, traffic and pedestrian movements shall be guided in a clear and positive manner by adequate signs, markers, pavement markings or by traffic control persons, whichever control measure or combination of control measures seems to be most appropriate.

The traffic control elements should be checked regularly by authorized persons to make sure that the traffic operations in the work areas are acceptable.

Work sites should be carefully checked to make sure that traffic controls are changed to suit changing construction conditions due to work staging and progress, or if an immediate improvement to the traffic control is needed.

A5.01.04 Urban Areas

The general principles outlined in this Office Manual are applicable to both rural and urban areas. The differences

between rural and urban situations warrant some separate treatment of traffic control requirements because of the lower speeds, higher volumes, limited maneuvering space, frequent turn and cross movements, pedestrian protection and encroachment by buildings found in an urban environment.

Due to the lower speeds and general space limitations in urban areas, fewer advance warning signs may be required. Standard size warning signs are generally acceptable. However, larger or additional signs may be needed on wide streets with high traffic volumes, especially when advertising displays and distracting backgrounds compete with a standard sized warning sign for the motorist's attention. The required advance distance for the placement of warning signs is generally much shorter in urban areas as compared to rural conditions.

The amount of street space taken up by construction and maintenance work should be no more than is absolutely necessary. However, this does not justify any failure to use sufficient warning and channelizing devices as may be required for public protection and guidance, or for the protection of the worker.

Pedestrian safety shall be given special consideration, particularly when the work area encroaches upon a sidewalk. Protective barricades, fencing, handrails or specifically designed passageways should be used in addition to warning and guiding devices to separate pedestrians from the traffic as well as from the work area.

A5.01.05 Definitions

Freeway

Any public highway with a continuous dividing median and a posted speed limit of 90 km/h or greater.

Moving Operations

Any maintenance activity which involves a work area that changes place continuously (eg. spraying or mowing operations), or involves frequent stops not exceeding 30 minutes duration (eg. patching, replacing street light, etc.).

Short Duration Work

Any daytime maintenance activity, construction project or utility work which requires a separate work area for less than one day in duration.

Long Duration Work

All construction and maintenance activities which require a separate work area greater than one day in duration.

Taper

The gradual narrowing of a lane using successive cones, markers or flexible drums which is intended to safely guide drivers into the next lane. The taper length is the distance of the section of roadway required to achieve full lane closure.

Tangent

A tangent section is the distance between the end of one taper and the beginning of the next taper.

Low Volume/High Volume Road

The volume of a roadway is normally expressed as a daily volume and includes the combined traffic in both directions. Low volume roads are defined as having volumes less than 3,000 vehicles per day. This figure can be obtained from the local road authority or can be estimated by counting the number of vehicles passing the work site in 3 minutes and multiplying this figure by 300 to get the daily volume.

Route Detour

A detour shall be signed when a driver is required to depart completely from his normal route and will be directed to follow an alternate route. The alternative route will be signed using a combination of the appropriate TC-10 directional signs. Prior to the closing of the roadway and the opening of a detour, a TC-65 "Road Closing Notice" sign shall be erected at strategically selected locations of the road at least one week in advance of the actual closing.

Roadside Diversion

A roadside diversion shall be signed to indicate a deviation of the normal roadway when work closes a section of a road and a short diversion is required to bypass the work area. The diversion shall be signed using a TC-9, TC-16 or other appropriate signs.

Portable Lane Control Signals

Portable lane control signals may be used as an alternative to Traffic Control Persons and are used only to stop vehicles intermittently when traffic must use a single lane in situations where the roadway is normally a two-way operation. Portable lane control signals shall comply in all respects with Section 125 and Regulation 478 of the Highway Traffic Act. (R.S.O. 1980)

Temporary Traffic Control Signals

Temporary traffic control signals are erected to control traffic at a crossing, such as a temporary roadway, a truck access route, pedestrian crossing, etc. The design specifications for temporary signals, which require prior approval by the M.T.O. or appropriate road authority, are those specifica-

tions which apply to permanent traffic control signals at signalized intersections (for details, see Part B of this manual).

A5.02 SIGNS

The signs in this section of the manual are designated by the letters "TC" followed by a number. The letters "TC" indicate the "Temporary Condition" series of signs.

All sign dimensions, including supports, positioning and pertinent erection details are given in metric measurement standards. The dimensions of signs shown in the Manual generally are standard sizes recommended for typical work zones on normal 2-lane roadways. The use of oversize dimensions, where illustrated, applies basically to freeways and to roadways having more than 2 lanes.

All temporary signs shall be removed or covered immediately after they are no longer applicable. Permanent standard signs must be in place at the completion of each work project. Signs pertaining to a moving operation, e.g., hot mix paving, shall be shifted along the roadway as the operation proceeds.

All sign patterns are available in full size from the M.T.O. Traffic Management and Engineering Office.

A5.02.01 Warning Signs (TC-1 to TC-39)

Warning signs for construction and maintenance work are the most important signs used to advise vehicle operators of specific hazards that may be encountered where roadway or utility construction or maintenance operations are underway.

Warnings for construction and maintenance shall generally be diamond shaped and shall have an orange reflectorized background with a black symbol and/or legend message and black sign border.

A5.02.02 Regulatory Signs (TC-41 to TC-46)

Regulatory signs impose legal obligations and/or restrictions on all traffic. While provisional control of traffic will ordinarily be accomplished through warning signs, there are temporary traffic situations in construction and maintenance where the use of regulatory signs becomes necessary.

Regulatory signs, with few exceptions, shall be rectangular in shape and shall have a white reflectorized background with a black painted symbol or legend message and a black sign border and must conform to the Manual of Uniform Traffic Control Devices for Ontario .

MTO BLANK NUMBER	SIZE	TYPE	THICKNESS
B-8	30 x 90 cm	Aluminum	1.6 mm
B-11	45 x 45 cm	Aluminum	1.6 mm
B-17	61 x61 cm	Aluminum	1.6 mm
B-18	60 x 60 cm	Galv. Steel	1.6 mm
B-20	60 x 90 cm	Galv. Steel	1.6 mm
B-23A	75 x75 cm	Galv. Steel	1.6 mm
B-25A	20 x 60 cm	Galv. Steel	1.6 mm
B-27	90 x 90 cm	Galv. Steel	1.6 mm
B-29	120 x 90 cm	Plywood	11.0 mm
B-30	120 x 120 cm	Plywood	11.0 mm
B-38A	90 x 180 cm	Plywood	19.0 mm
B-43	120 x 180 cm	Plywood	19.0 mm
B-44	120 x 210 cm	Plywood	14.0 mm
B-45	120 x 240 cm	Plywood	19.0 mm
B-52	75 x 60 cm	Aluminum	1.6 mm
B-54	90 x 240 cm	Plywood	19.0 mm

A5.02.03 Guide Signs (TC-61 to TC-75)

Guide signs are required at construction and maintenance work areas to guide traffic around or through work areas or to provide information to vehicle operators relative to detours, directions, types of construction and other information considered beneficial or essential to the motorist.

Guide signs, with few exceptions, shall have a rectangular shape with the longer dimension being horizontal. They shall have a white reflectorized background with a black painted legend and a sign border, except as noted in the sign description for Contract Identification signs.

Guide signs indicating road names and directions generally have a reflectorized green background with a white legend and border.

A5.03 SIGN SPECIFICATIONS

A5.03.01 Standard of Reflectorization

The minimum standard required for retro-reflective sheeting on orange coloured construction signs, markers and barriers is that described as "Reflectivity Level II" (Engineering Grade) under section 3 "Classification" of the Canadian Government Specification Standard 62-GP-11, as amended, dealing with retro-reflective marking material.

As of January 1, 1995 the following devices shall have a minimun reflectivity standard of "Reflectivity Level 1" (hi-intensity) Canadian Government Specification Standard 62-GP-11, as amended:TC 3, TC 4, TC 7, TC 18, TC 43, TC 46, TC 52, TC 53, TC 54. It is recommended that after January 1, 1991 only signs with "Reflectivity Level 1" (hi-intensity) be used for the above mentioned devices.

A5.03.02 Sign Blanks and Specification Details

The preceding table covers the sign dimensions, sign material and material thickness of the more common signs through assigned Blank Numbers. (Refer also to Sign Schedule).

Fiberglass reinforced plastic or other material may be substituted for some aluminum or galvanized steel sign blanks. The use of plastic or other material as sign blank material should be subject to approval by the road engineer or superintendent.

A5.04 ERECTION OF SIGNS

Signs generally shall be located on the right side of the roadway. Supplementary signs may be located on the left side of the roadway where the median is wide enough to accomodate the signs. They may also be used on multilane one-way streets, and in situations where experience has shown that drivers fail to see the primary signs.

Adjustments to the height and distance requirements may have to be made to allow placement of signs in constricted urban areas.

NOTE: Signs mounted in or on weighted barrels (e.g. 45 gallon steel drums) shall not be used.

A5.04.01 Positioning of Signs

Signs shall be placed in positions where they will convey the message most effectively without restricting lateral clearance or sight distance, and at advance distances that allow sufficient response time. They shall be mounted approximately at right angles to the direction of travel they are to serve.

A5.04.02 Freeway

Signs for construction projects shall be erected on either direct buried posts or mounted on bases with sufficient ballast that they will remain in position for the duration of the project.

Signs 120 cm of less in width shall be erected on single posts to a height of between 1.5 to 2.5 m above the travelled portion of the roadway to the bottom edge of the sign. Signs exceeding 120 cm in width shall be erected on two posts at a general height of 1.5 m above the travelled portion of the roadway to the bottom edge of the sign, with the exception of the diamond shaped two-post sign which shall be 1 m minimum.

Signs erected for daytime maintenance projects are usually positioned on the usable portion of the shoulder. The supports for these signs should be constructed in such a manner that they will not be a hazard to errant vehicles, yet be sufficient enough to remain upright. Signs 90 x 90 cm or larger shall be erected to a height of between 1 m minimum to 2.5 m above the roadway. Smaller signs (75 x 75cm) shall be erected to a height of between 1.5 m minimum to 2.5 m above the roadway.

A5.04.03 Non Freeway

Signs for construction projects shall be erected on direct buried posts or mounted on bases with sufficient ballast that they will remain in position for the duration of the project.

Signs 120 cm or less in width shall be erected on single posts to a height of between 1.5 to 2.5 m above the travelled portion of the roadway to the bottom edge of the sign. Signs exceeding 120 cm in width shall be erected

on two posts at a general height of 1.5 m above the travelled portion of the roadway to the bottom edge of the sign, with the exception of the diamond shaped two-post sign which shall be 1 m minimum.

Signs erected for maintenance projects are usually positioned on the usable portion of the shoulder. The supports for these signs should be constructed in such a manner that they will not be a hazard to errant vehicles, yet be sufficient enough to remain upright. Signs 90 x 90 cm or larger shall be erected to a height of between 1 m minimum to 2.5 m above the roadway. Smaller signs (75 x 75cm) shall be erected to a height of between 1.5 m minimum to 2.5 m above the roadway.

A5.04.04 Low Mounted Portable Sign Stands

Low Mounted Portable Sign Stands (e.g. "A-Frame Type") may be used only for the TC-2A, TC-2B, TC-21 or the TC-26. Signs mounted on these stands shall have flags attached to them to bring the drivers attention to the message on the sign (see A5.06 for proper flag mounting). The sign bases must be of sound material as to adequately support the sign, however, the base should not be appreciably wider than the sign. Ballast, if required, will prevent the sign from blowing over in the wind. The ballast must not be material which can be hazardous if struck by a vehicle such as a piece of steel or block of concrete.

A5.05 MAINTENANCE OF SIGNS

All signs for work projects shall be in positions where they can be readily seen by the travelling public at all times. Intersections shall be identified with standard turn-off signs. Route markers shall be located along the roadway to ensure route continuity. **Stop signs must be maintained at all times.**

On any work project, the supervisor is responsible for keeping a record of all traffic control devices used on the site. For major projects, a separate field book should be maintained. The daily status and the times of any moves should be recorded. As soon as possible after any accident the status of signs in the area concerned and the time of inspection shall be recorded and any necessary measurements or photographs taken.

Signs that have been damaged or defaced should be replaced immediately.

Dirty signs result in poor visibility, particularly at night, therefore, **ALL** signs shall be maintained in a clean, legible and good working condition in order to be effective.

A5.06 FLAGS

Flags are not to be used as channelizing devices or traffic control devices, but shall be used in conjunction with low mounted portable sign stands to enhance their visibility. Flags shall be 1.5 to 2.5 m above roadways.

A5.07 ROUTE DETOURS

A route detour shall be signed when a driver is required to depart completely from his normal route and will be directed to follow an alternative route. The alternative route will be signed using a combination of the appropriate TC-10 directional signs.

Prior to the closing of a roadway and the opening of a route detour, a TC-65 "Road Closing Notice" sign shall be erected at strategically selected locations of the road at least one week in advance of the actual closing. At the same time information regarding the closing may be given to local radio and T.V. stations and local newspapers. Notices should also be distributed to affected households and businesses advising them of the upcoming disruption in their area.

A pre-construction planning meeting should be held with representatives of Police, Ministry of Labour, Fire and Ambulance departments as well as any Transit Authority to advise them of the situation and to allow them to assess how this will affect their functions and responsibilities.

Before a route or a temporary detour is opened to the public all signs pertinent to the condition shall be erected in their proper positions. Construction Ahead signs shall be erected when work first commences. The Detour signs, etc., shall not be exposed until the detour is actually ready to open.

A5.08 CHANNELIZING DEVICES AND BARRICADES

The functions of channelizing devices are to warn and alert vehicle operators to hazards created by construction or maintenance activities in or near the travelled way, and to guide and direct traffic safely past the hazards. Channelizing devices as used herein include cones, construction markers, flexible drums, pavement markings and New Jersey type concrete barriers.

A5.08.01 Channelizing Devices

The single most important element within the system of traffic control devices used in construction or maintenance work areas is the transition taper for full lane closure or for other reductions in the pavement width. An inadequate taper will almost always produce undesirable traffic operations with resulting congestion and the possibly of accidents through the area. Vehicles and equipment shall not be kept in the taper area.

The minimum desirable taper length for common offset widths has been calculated for various approach speeds, and subsequent taper length values as well as cone or

marker spacings can be read from the appropriate Table (A, B or C). These tables will also provide general guidance as to speed related distances and spacing relationships for the placement of general advance warning signs.

Taper alignments for channelization by cones, markers or flexible drums may get out of their original alignment due to being struck by vehicles or moved by construction activities. It is therefore necessary to patrol the channelization at frequent intervals to ensure it is functioning properly. On long term construction projects repositioning of channelizing devices can be accomplished expeditiously if the original taper alignment has been indicated by paint markings or the application of removable marking tape on the pavement.

A5.08.02 Barricades

The primary function of barricades is to delineate excavation or construction areas in or near the travelled portion of the roadways, and to block off a portion or all of the street where partial or full road closures become a neccessity.

On divided high-speed, high volume roadways, vehicular penetration into work sites can be prevented by the use of "New Jersey" type barrier. Although barriers may serve the additional function of channelizing traffic, their use should be determined by the protective requirements of the location rather than the channelizing needs. Temporary pavement edge lines as well as reflective delineation should be used with "New Jersey" type concrete barriers.

A5.09 TEMPORARY PAVEMENT MARKINGS

Temporary Pavement Markings shall be used where traffic is diverted from normal paths, and where guidance by channelizing devices, delineation or warning signs does not clearly indicate the required vehicle path. Conflicting pavement markings which might inadvertently lead motorists from the intended path, should be obliterated to prevent confusion.

Temporary pavement markings should also be considered on projects with partial pavement removal (milling). These markings should as closely as possible reflect the pavement markings which had been removed.

When a temporary roadway is constructed to bypass a closed portion of highway, appropriate reflectorized pavement markings shall be placed on the approach to, and throughout the length of the hard surfaced temporary roadway. At locations where the temporary roadway is of relatively short distance, temporary traffic paint or pressure sensitive marking tape which can be supplemented by raised pavement markers, may be used to provide short-term expendable pavement markings.

Temporary pavement markings shall comply with the standards prescribed in the Manual of Uniform Traffic Control Devices for Ontario. Any deviations from the standards, for economical or other reasons, are subject to approval by the road authority, and such deviations must not compromise the safety or the proper guidance of the traffic flow.

Temporary pavement markings are normally used in combination with appropriate warning signs, channelizing devices and delineation to mark the intended vehicle path that traffic is to follow.

A5.10 SPEED CONTROL IN CONSTRUCTION AREAS

It has generally been found that control of traffic speeds by imposing unwarranted regulatory speed limits has not been very effective. The majority of drivers disregard posted speed limits if the construction activities or hazards encountered are not severe enough to warrant such lower speeds. Emphasis shall be put on the need to design the travel way through the construction zone at a design speed that is equal to or as close as possible to that of the approaches to the construction area.

There are many types of maintenance and construction projects where a reduction of the normal speed limit is not required. A basic guideline to follow is not to attempt to reduce speeds if there is not a good reason for it. If there is a good reason for reducing speed which may not be readily apparent to the motorist, then he should be informed of the reason for a speed reduction through advance signing. In many cases, advisory speed limit signs (orange and black) together with narrower passageways through the work site have been more effective in reducing traffic speeds than by posting legal speed controls. All reduced speed zones should be used in a flexible manner to reflect the changing conditions within the construction or maintenance zone.

For M.T.O. construction projects requiring traffic control, construction speed zones may be established. If construction speed zones are to be established it is necessary to erect Construction Zone Begins and Ends signs (TC-41A and TC41B) in advance and beyond the construction speed zone. Both regulatory and advisory speed limit signs can be used on different portions of the same contract, however, regulatory speed limit signs shall be erected only when the appropriate police authority has been informed and is intending to provide enforcement. Otherwise, only advisory signs should be used and all existing regulatory signs within the construction speed zone shall be covered or removed for the duration of the construction project.

On a highway, it is permissible to have different speed limits in either direction of travel. If construction involves only one side of a highway, the speed limit may be lowered in the effected direction of travel but remain unaltered in the opposite direction.

A5.11 TRAFFIC CONTROL PERSONS (TCP)

Traffic Control Persons (TCP's) may be used at work sites to stop traffic intermittently as necessitated by work progress, or to maintain continuous traffic flow past the work site at reduced speeds to protect the work crew. They shall be positioned in a manner which will not conflict with other traffic control devices such as stop signs or traffic signals. Where traffic must use a single lane in situations where the roadway is normally a two-way operation, TCP's may be used to control a one-way traffic movement so that vehicles will not be moving in opposing directions at the same time. On sections where TCP's are not in sight of each another, a third TCP, or some other means of communication, is required to relay instructions to the TCP at either end.

The responsibilities prescribed by the Occupational Health and Safety Act make it mandatory that adequate safety precautions are taken to protect the TCP from particular hazards to which the TCP may be exposed. This includes not only personal protective clothing, equipment and devices, but extends to appropriate protective measures against dangers and risks imposed by vehicular traffic. This safety aspect must receive prime consideration during the planning stages of traffic control. The appropriate equipment, dress requirements and the duties of a TCP are outlined in this Manual and also in the Handbook for Construction Traffic Control Persons which is published by the Construction Safety Association of Ontario. Information about publications and training courses for Traffic Control Persons can be obtained from the Construction Safety Association of Ontario, 74 Victoria Street, Toronto, Ontario, M5C 2A5, (416) 366-1501 or 1-800-387-0847.

Hand Signalling Devices

The standard TC-22 traffic control sign with extension handle shall be used by TCP's to direct traffic. **The use of flags is PROHIBITED!**

The standard TC-21 "TCP Ahead" sign shall be positioned in advance of the TCP at a distance determined from Table A or B.

TRAFFIC CONTROL PERSON(S)
(WORKERS DIRECTING TRAFFIC BY MEANS OF SIGNS)
MAY BE USED
AS OUTLINED IN CHART BELOW

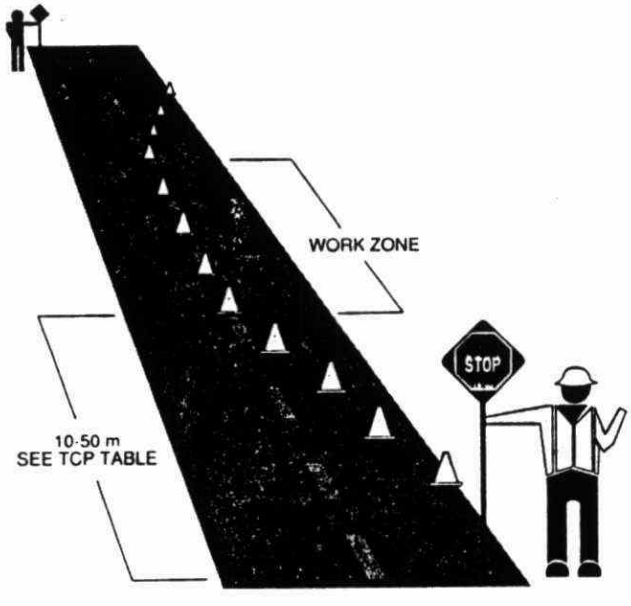
CONDITIONS	POSTED SPEED 60km/h OR LESS, ONE LANE OR REDUCED TO ONE LANE IN EACH DIRECTION	POSTED SPEED EXCEEDS 60km/h, ONE LANE OR REDUCED TO ONE LANE IN EACH DIRECTION	ANY SPEED MORE THAN 1 LANE EACH DIRECTION
WORKERS ON PUBLIC WAY	YES	YES	NO
CONSTRUCTION VEHICLES CROSSING ROADWAY	YES	DIRECT CONSTRUCTION TRAFFIC ONLY NOT PUBLIC TRAFFIC	NO
CONSTRUCTION VEHICLES ENTERING A ROADWAY	YES	DIRECT CONSTRUCTION TRAFFIC ONLY NOT PUBLIC TRAFFIC	NO

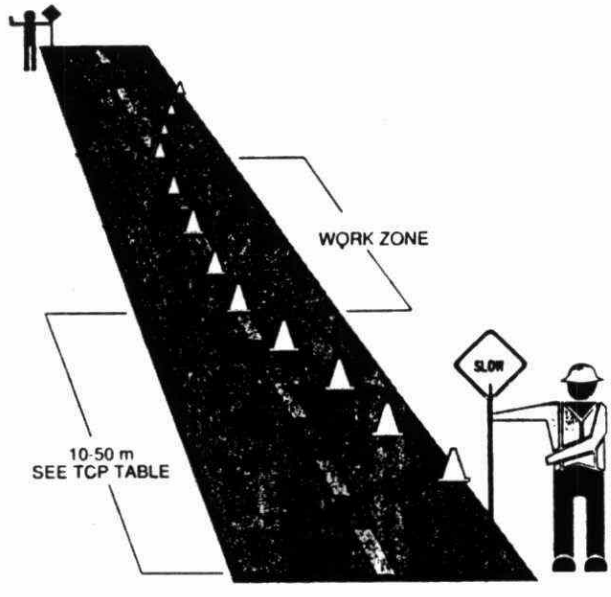
NOTE:See the OCCUPATIONAL HEALTH AND SAFETY ACT AND REGULATIONS FOR CONSTRUCTION PROJECTS. R.S.O. 1980 (ACT 13 and 14) (Regulation 691, Sections 24, 25, 26, and 27)

TRAFFIC CONTROL PERSONS

T.C.P. TABLE

POSTED SPEED	60 km/h OR LESS, ONE LANE OR REDUCED TO ONE LANE IN EACH DIRECTION		70 km/h OR GREATER, ONE LANE OR REDUCED TO ONE LANE IN EACH DIRECTION	
TRAFFIC VOLUME	LOW	HIGH	LOW	HIGH
DISTANCE OF T.C.P. FROM WORK ZONE	10 - 15 m	20 - 30 m	30 - 40 m	40 - 50 m





Location or Position of TCP

One consideration in selecting the TCP's position is to maintain colour contrast between the TCP and background. TCP's shall be clearly visible to the approaching motorist at all times.

WHERE TO STAND

1. Stand just outside the lane of traffic.
2. Stand from 10 m to 50 m (See TCP Table) from the end of the work zone, so as to be able to protect worker or equipment.
3. Stand where TCP can be seen and he can see for at least 150 m.

HOW TO SLOW TRAFFIC

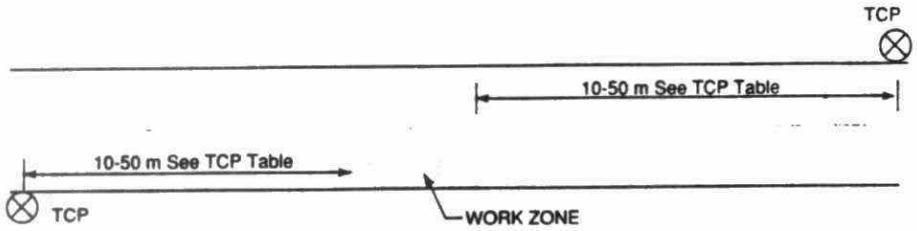
1. To slow traffic extend traffic control sign into lane of oncoming traffic and slowly move sign back and forth.
2. To keep traffic from coming to a full stop, wave traffic forward with other hand.

WEAR PROTECTIVE CLOTHING

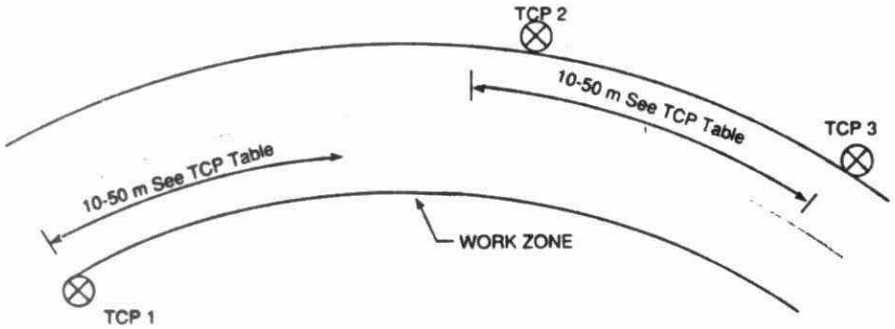
1. Safety Hat
2. A vest which shall be reflective, fluorescent and coloured blaze orange or red.
3. Safety Footwear

POSITIONING OF TRAFFIC CONTROL PERSONS

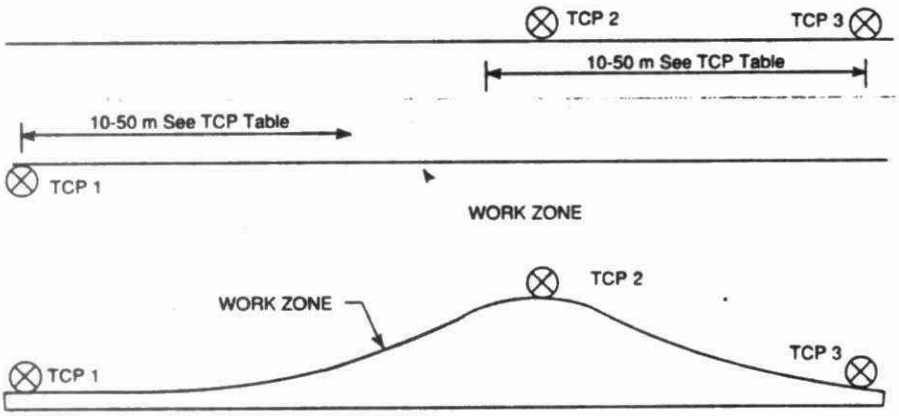
STRAIGHT ROAD



CURVE



HILL



NOTE:

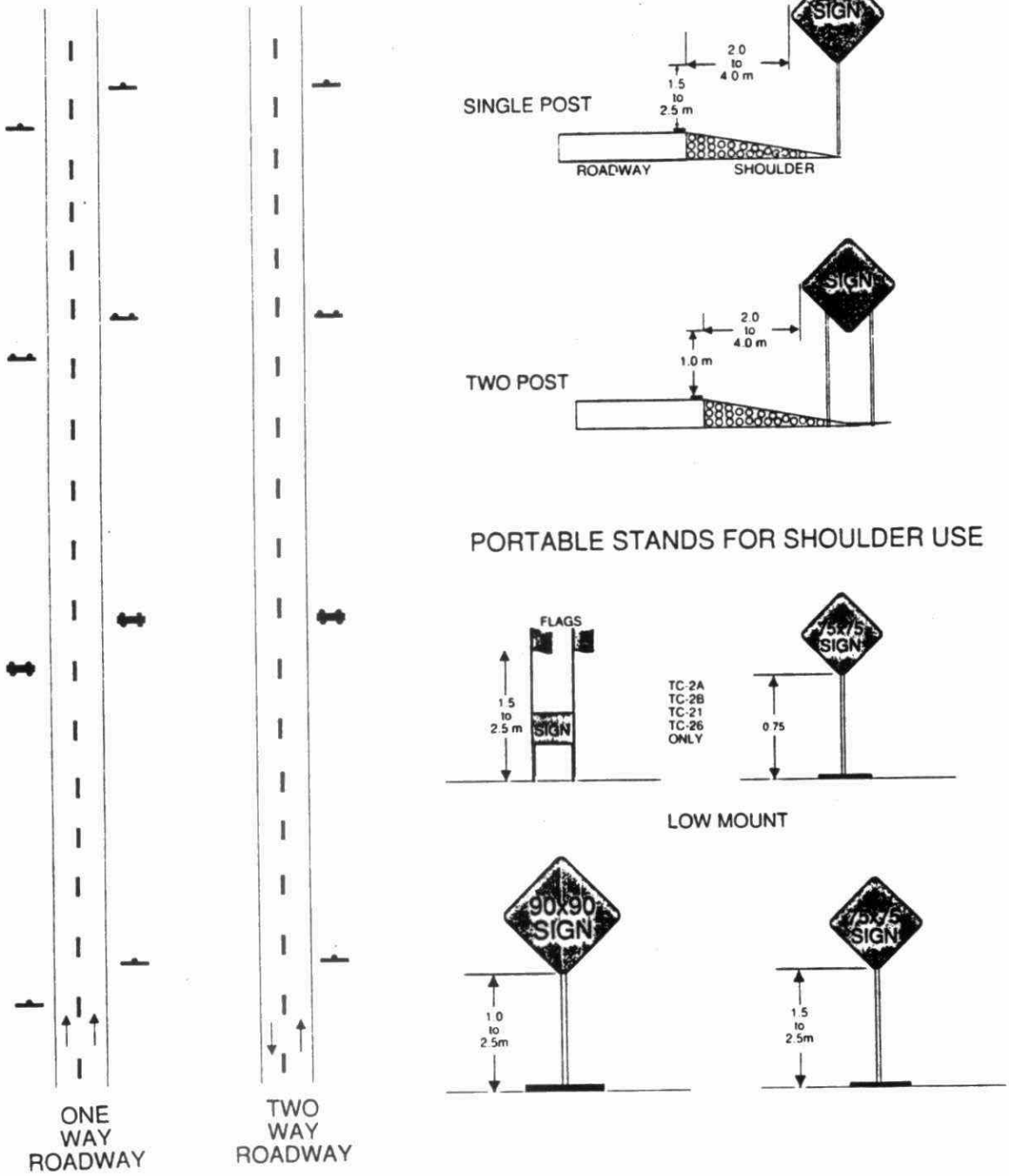
ON CURVES AND HILLS, THREE TCP'S OR SOME OTHER MEANS OF COMMUNICATION ARE REQUIRED. THE DUTY OF TCP NUMBER 2 IS TO RELAY SIGNALS BETWEEN TCP NUMBER 1 AND NUMBER 3.

SIGN SCHEDULE

In the following sections are sign illustrations for the most common type of roadway work operations. In addition to a detailed description of the purpose and proper use of each sign, a summary of the most commonly used signs in roadway work operations is included at the beginning of this section for easy reference.
Full scale patterns for the signs indicated here in are available through the Traffic Management and Engineering Office, Ministry of Transportation , Downsview, Ontario. M3M 1J8.

NOTE: As of January 1, 1995 the following devices shall have a minimum reflectivity standard of "Reflectivity Level 1" (hi-intensity) Canadian Government Specification Standard 62-GP-11, as amended: TC 3, TC 4, TC 7, TC 18, TC 43, TC 46, TC 52, TC 53, TC 54. It is recommended that after January 1, 1991 only signs with "Reflectivity Level 1" (hi-intensity) be used for the above mentioned signs.

TYPICAL SIGN PLACEMENT



THE OBJECTIVE IS TO PLACE THE SIGNS IN DRIVER'S CONE OF VISION

CONSTRUCTION AHEAD
TC-1 (90 x 90)
TC-101 (120 x 120)

CONSTRUCTION AHEAD (1 km)
TC-1A (90 x 90)
TC-101A (120 x 120)

ROAD WORK
TC-2A (60 x 60)

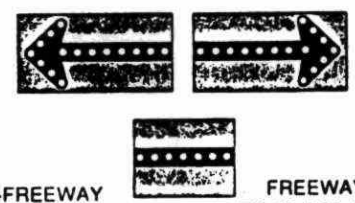
ROAD WORK
TC-2B (75 x 75)

RIGHT LANE CLOSED AHEAD
TC-3R (75 x 75)
TC-103R (90 x 90)

LEFT LANE CLOSED AHEAD
TC-3L (75 x 75)
TC-103L (90 x 90)

DETOUR AHEAD
TC-5 (90 x 90)
TC-105 (120 x 120)
TC-1105 (180 X 180)

DETOUR AHEAD (2 km)
TC-6 (90 x 90)
TC-106 (120 x 120)
TC-1106 (180 X 180)

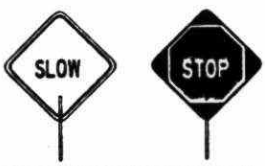


NON-FREEWAY
TC-12 (60x75)

FREEWAY
TC-12 (120x210)

PAVEMENT ENDS
TC-13 (90 x 90)

TRAFFIC CONTROL
PERSON AHEAD
TC-21 (60 x 60)

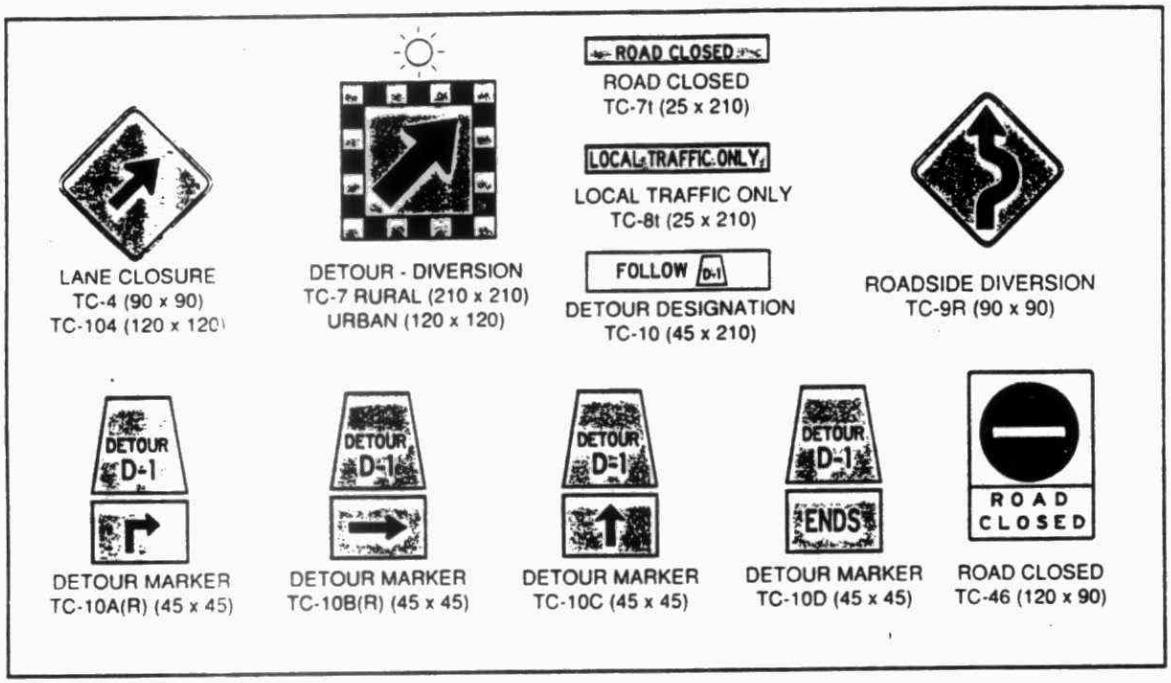


TRAFFIC CONTROL SIGN
TC-22 (45 x 45)

SIGNALS AHEAD
TC-23 (60 x 60)

TRUCK ENTRANCE
TC-31R (90 x 90)

TRUCK ENTRANCE
TC-31L (90 x 90)



LANE CLOSURE
TC-4 (90 x 90)
TC-104 (120 x 120)

DETOUR - DIVERSION
TC-7 RURAL (210 x 210)
URBAN (120 x 120)

ROAD CLOSED
TC-7t (25 x 210)

LOCAL TRAFFIC ONLY
TC-8t (25 x 210)

DETOUR DESIGNATION
TC-10 (45 x 210)

ROADSIDE DIVERSION
TC-9R (90 x 90)

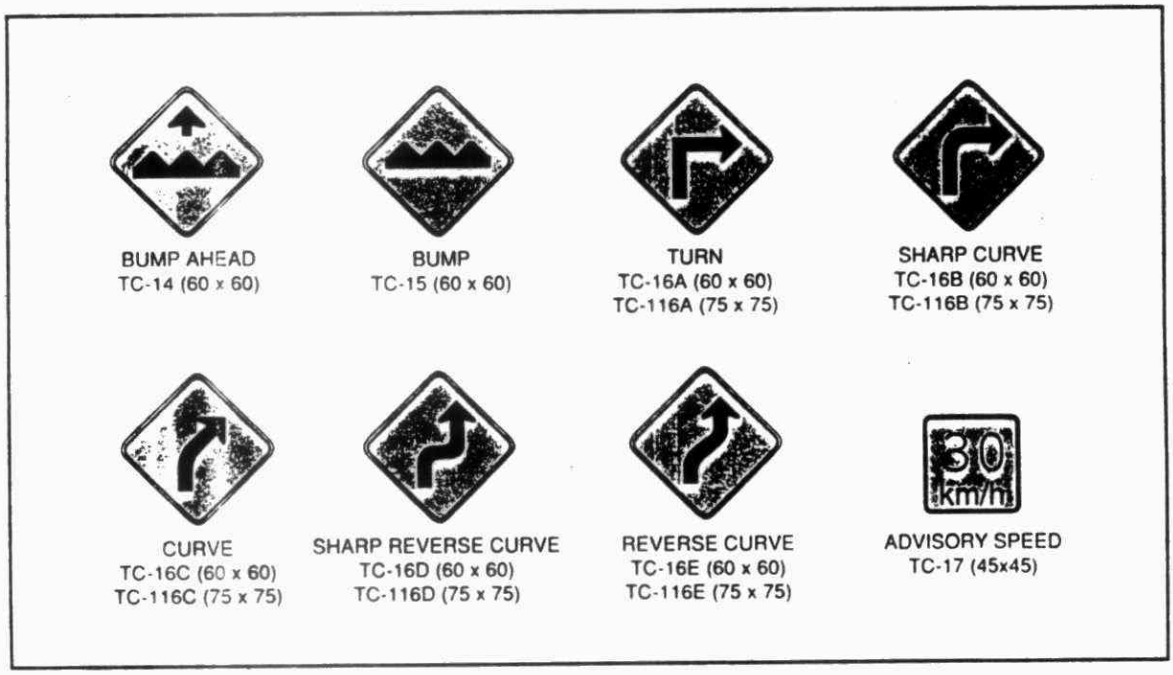
DETOUR MARKER
TC-10A(R) (45 x 45)

DETOUR MARKER
TC-10B(R) (45 x 45)

DETOUR MARKER
TC-10C (45 x 45)

DETOUR MARKER
TC-10D (45 x 45)

ROAD CLOSED
TC-46 (120 x 90)



BUMP AHEAD
TC-14 (60 x 60)

BUMP
TC-15 (60 x 60)

TURN
TC-16A (60 x 60)
TC-116A (75 x 75)

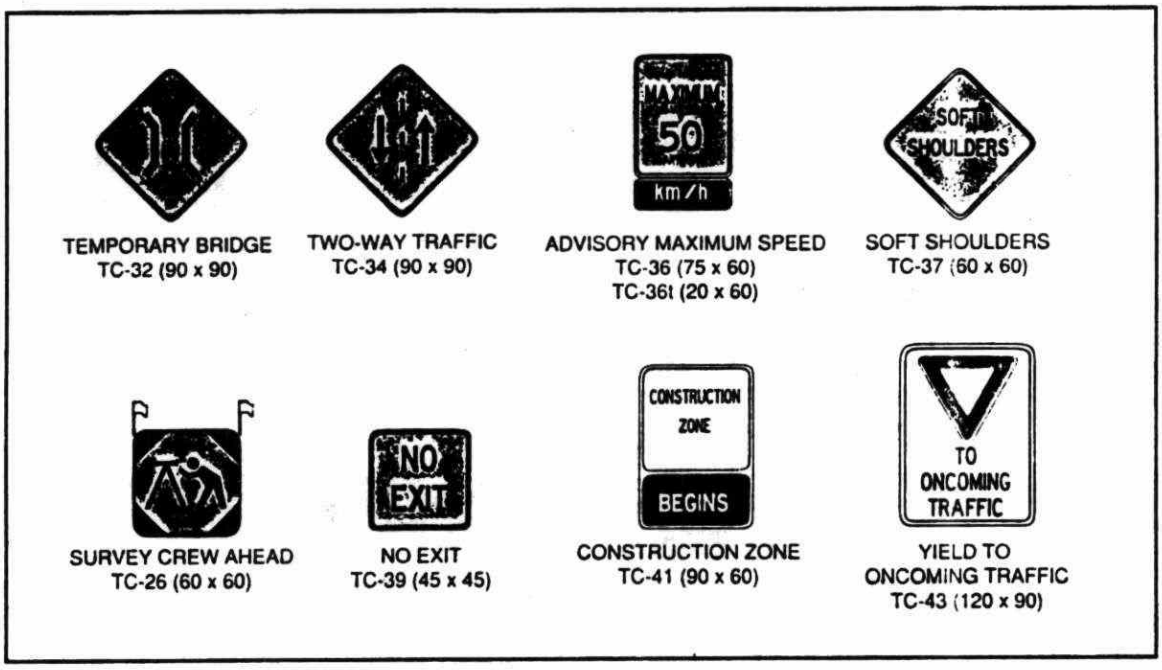
SHARP CURVE
TC-16B (60 x 60)
TC-116B (75 x 75)

CURVE
TC-16C (60 x 60)
TC-116C (75 x 75)

SHARP REVERSE CURVE
TC-16D (60 x 60)
TC-116D (75 x 75)

REVERSE CURVE
TC-16E (60 x 60)
TC-116E (75 x 75)

ADVISORY SPEED
TC-17 (45x45)



TEMPORARY BRIDGE
TC-32 (90 x 90)

TWO-WAY TRAFFIC
TC-34 (90 x 90)

ADVISORY MAXIMUM SPEED
TC-36 (75 x 60)
TC-36t (20 x 60)

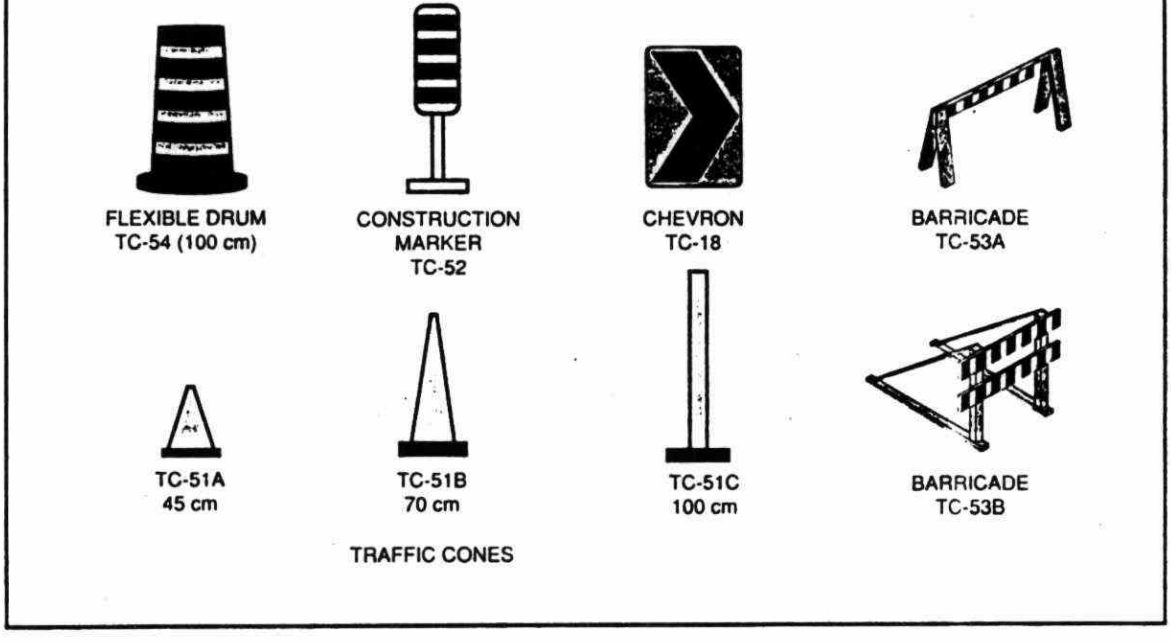
SOFT SHOULDERS
TC-37 (60 x 60)

SURVEY CREW AHEAD
TC-26 (60 x 60)

NO EXIT
TC-39 (45 x 45)

CONSTRUCTION ZONE
TC-41 (90 x 60)

YIELD TO ONCOMING TRAFFIC
TC-43 (120 x 90)



FLEXIBLE DRUM
TC-54 (100 cm)

CONSTRUCTION MARKER
TC-52

CHEVRON
TC-18

BARRICADE
TC-53A

TC-51A
45 cm

TC-51B
70 cm

TC-51C
100 cm

BARRICADE
TC-53B

TRAFFIC CONES

A5.21 "CONSTRUCTION AHEAD" Signs (TC-1) (TC-1A)

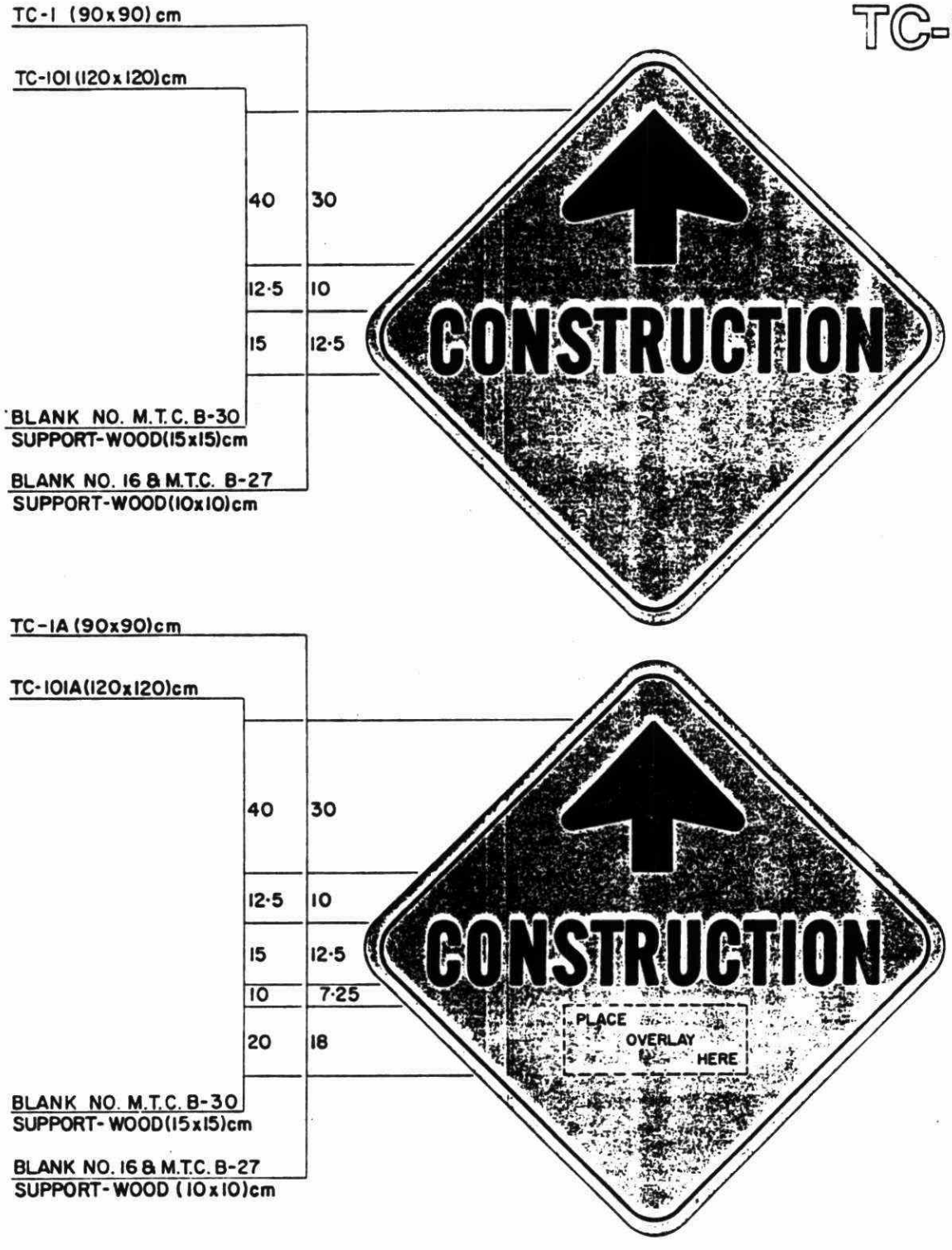
The TC–1 sign shall be used to provide advance warning of a major construction zone. The oversized TC–101 sign shall be used on roadways having more than two lanes, except in urban areas.

On a divided highway, two TC–101 signs shall be erected, one on each side of the approaching lanes, with the exception that the TC–1 sign may be used in place of the oversized TC–101 sign on a narrow median.

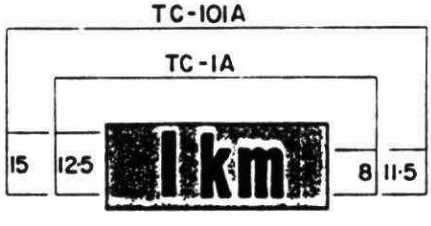
The TC–1 sign shall be erected in advance of the construction zone at a distance which is to be determined by reference to the appropriate Table (A, B or C). In urban areas (60 km/h or less) one sign is sufficient. In rural areas a TC–1A (TC–101A) sign shall be added at a distance of 1 km in advance of the construction zone. For freeways a TC–101A sign shall be erected at distances of 1 km and 2 km from the work area.

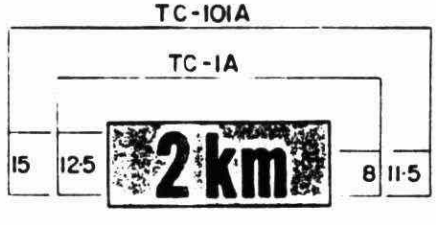
On minor intersecting roads the TC–1 sign shall be erected in advance of the construction site on the major roadway at a distance that is to be established by referring to the appropriate Table (A, B or C). On major intersecting roads the TC–101 sign shall be preceded by a TC–101A sign 1 km in advance of the construction area on the crossing road.

TC-1, 1A



OVERLAYS





TC-2A, 2B

TC-2A (60x60)cm
BLANK NO.4 & M.T.C. B-17
SUPPORT - BRACKET

TC-2B (75x75)cm
BLANK NO. 15 & M.T.C. B-23a
SUPPORT-WOOD (10x10)cm

A5.22 "ROAD WORK" Sign (TC-2A, TC-2B)

The TC–2A sign shall be used as indicated in the typicals at all times when maintenance repairs or minor construction work is carried out. The sign shall be erected on a portable stand and shall be displayed only during the times when repair or minor construction work is in progress. It shall be placed face down or removed when activities are temporarily suspended such as at lunch time or at the close of the day.

The TC–2B sign shall be erected when the maintenance or minor construction activity extends over longer periods of time, and is of a more stationary nature. It may also be used at intermediate locations on long construction areas to set apart certain road sections having a higher degree of construction activities than observed in other intersections.

The signs shall be located on the shoulder or at the curb in full view of approaching traffic. The signs shall be erected at a distance from the work site to be established by referring to Table A. When the TC–21 "Traffic Control Person Ahead" sign is used, the "Road Work" signs shall be located at the same distance as defined above, in advance of the TC–21 sign.

TC-3

TC-3L (75x75) cm
BLANK NO. 15 & M.T.C. B-23a
SUPPORT - WOOD (10x10) cm

TC-103L (90x90) cm
BLANK NO. 16 & M.T.C. B-27
SUPPORT - WOOD (15x15) cm

TC-3R (75x75) cm
BLANK NO. 15 & M.T.C. B-23a
SUPPORT - WOOD (10x10) cm

TC-103R (90x90) cm
BLANK NO. 16 & M.T.C. B-27
SUPPORT - WOOD (15x15) cm

TC-3tL (60x60) cm
BLANK NO. 4 & M.T.C. B-17

TC-3tR (60x60) cm
BLANK NO. 4 & M.T.C. B-17

A5.23 "LANE CLOSED AHEAD" Sign (TC-3L, TC-3R)

The TC-3 sign shall be used to provide advance warning of a lane closed for road work. The oversized TC-103 sign shall be used on roadways with more than two lanes, although the smaller TC-3 signs may be used where they are required only for short term daytime work operations in all urban work operations.

On a divided highway, two TC-103 signs shall be erected, one on each side of the approaching lanes, with the exception that the TC-3 sign may be used in place of the oversized TC-103 sign on a narrow median.

The signs shall be erected at a distance in advance of the TC-4"Lane Closure" sign which is to be established by reference to the appropriate Table (A, B, or C).

The signs shall be used for "Right" and "Left" lane closures with the appropriate reversal of the sign symbol layout. Where lane closures involve more than one lane, use signs twice in accordance with the staging of 2 lane closures.

The educational tab sign (TC-3tL) or (TC-3tR) shall be used until such time that the meaning of the symbol has become sufficiently familiar to the driving public.

TYPICAL SIGN LAYOUTS

The signing for the most common types of roadway work operations are presented in this section. They show the type, sequence, and longitudinal placement of the standard signing required for the various construction or maintenance situations.

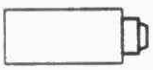
The Section is divided into three parts:

A - Short Duration Work

B - Long Duration Work

C - Freeway Conditions

A legend of the symbols frequently used in the sign layouts for all three parts is presented below:

LEGEND	
	CONE, TC-52 OR TC-54
	SIGN
	TRAFFIC CONTROL PERSON (TCP)
	TRUCK OR MAINTENANCE VEHICLE
	FLASHING AMBER LIGHT
	VEHICLE MOUNTED 360° AMBER LIGHT
	WORK ZONE
	PORTABLE LANE CONTROL SIGNAL
	BARRICADES

A — SHORT DURATION WORK

Short duration work includes any daytime maintenance activity, construction project or utility work which requires a separate work area for less than one day in duration. The following section presents the sign layouts for the most common types of short duration work operations (non-freeway conditions). They show the type, sequence and placement of the standard signing of the various work situations.

A1 Shoulder Operations

A1.1 Shoulder Operations (Less than 30 minutes)
A1.2 Shoulder Operations (30 minutes or greater)

A2 Moving Operations

A2.1 Moving Operations
A2.2 Moving Operations — Multi Lane Roadway

A3 Roadside Work

A3.1 Low Volume Unpaved Roads
A3.2 Encroachment in Right Lane
A3.3 Parking Lane Occupied

A4 Lane Closure

A4.1 Two Lane Roads (TCP'S)
A4.2 Right Lane Closed — Multi Lane Road
A4.3 Left Lane Closed — Multi Lane Road
A4.4 Centre Two-way Left Turn Lane Closed

A5 Intersections

A5.1 One Lane Closed (Near Side)
A5.2 One Lane Closed (Far Side)
A5.3 Two Lane Intersections — One Lane Closed (Near Side)
A5.4 Two Lane Intersections — One Lane Closed (Far Side)
A5.5 Two Lane Roadways — Work in Intersection
A5.6 Four Lane Roadway — One Lane Closed (Far Side)

A6 Sidewalk Detour Onto Roadway

A7 **Zone Painting Operations**

A7.1 Coning Paint — Two Lane Road
A7.2 Coning Paint — Multi Lane Road
A7.3 Non-Coning Paint — Two Lane Road
A7.4 Non-Coning Paint — Multi Lane Road
A7.5 Turn Arrows
A7.6 Stoplines and Crosswalks
A7.7 Intersections — Left Lanes Closed
A7.8 Intersections — Right Lanes Closed

A8 **Surveying**

A8.1 Instrument on Shoulder
A8.2 Instrument on Centreline (Tangent)
A8.3 Work on Centreline (Curve or Hill)
A8.4 Work in Intersections

TABLE A — SHORT DURATION WORK

		REGULATORY SPEED LIMIT			
		60 km/h or less		70 km/h or greater	
*	Traffic Volume	Low Vol.	High Vol.	Low Vol.	High Vol.
1a	Taper Length for Full Lane Closure (m)	10-15	20-30	30-40	40-50
1b	Taper Length for Roadside Work (m)	3-5	5-7	7-10	10-12
2	Maximum Distance Between Markers (m)	4-6	4-6 (use at least 4 markers)	8-10	8-10
3	Minimum Tangent Between Tapers (m)	30	30	60	60
4	Distance Between Construction Signs (m)	20-30	20-30	50-60	50-60
	CONDITION	TC-2A Required ?			
5	-Visibility less than 150 metres.	Yes	Yes	Yes	Yes
	-Visibility 150 metres or greater	No	No	No, if a TC-12 is used.	Yes

- Low volume is defined as an average daily traffic volume less than 3000 vehicles per day (combined traffic for both directions). This figure can be obtained from the local road authority or can be estimated by counting the number of vehicles passing the work site in 3 minutes and multiplying this figure by 300.

  Example: 3 minute count of combined traffic in both directions: 20 vehicles X 300 = 6000 vehicles per day. (This would be a high volume road).

- **NOTES :**
  When the 85th percentile speed is known, it should be used instead of the posted regulatory speed limit.

FIGURE A3.3 ROADSIDE WORK (PARKING LANE OCCUPIED)

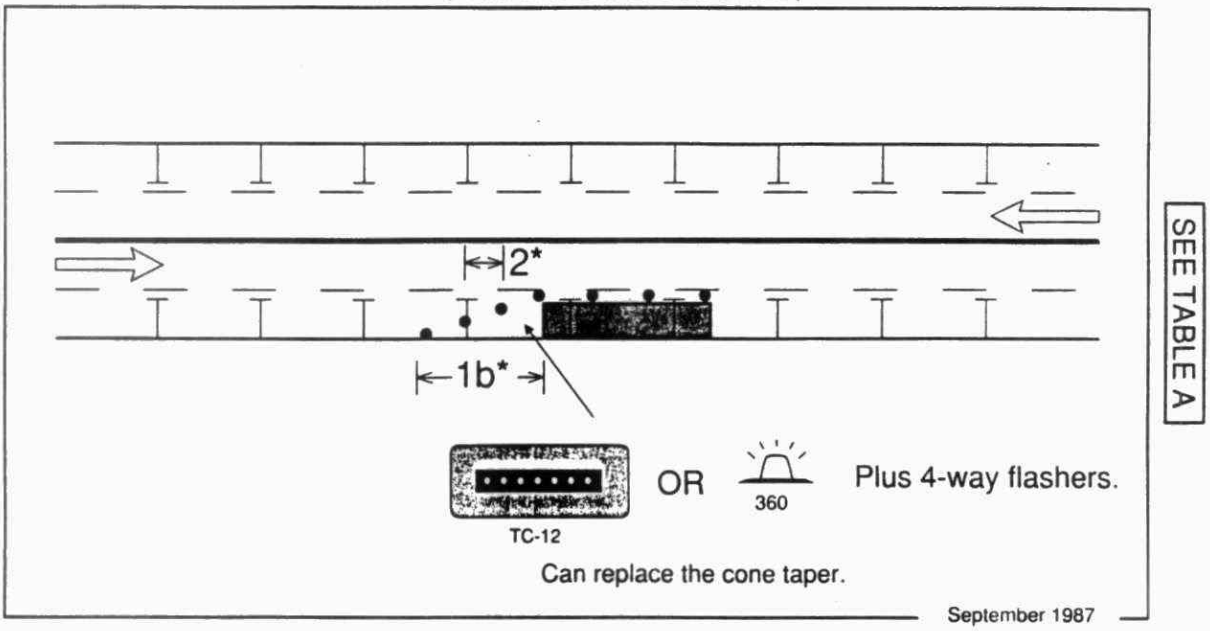
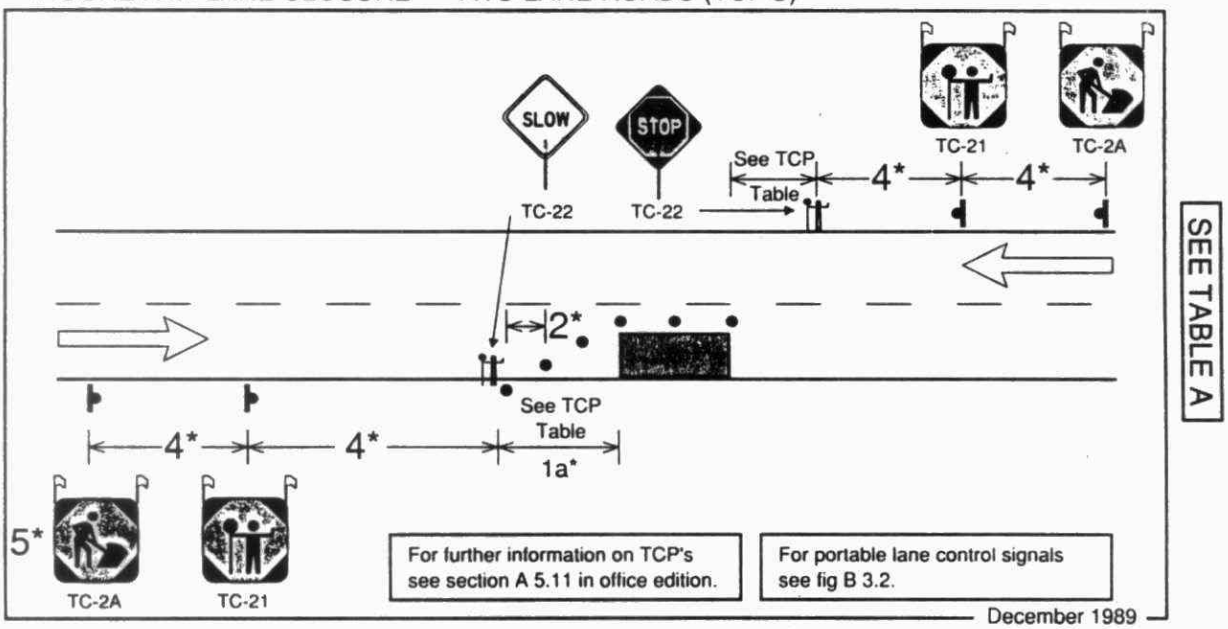


FIGURE A4.1 LANE CLOSURE — TWO LANE ROADS (TCP'S)



- For low-volume roads, and short length work areas (less than 150m) if no TCP is on duty, a TC-43 (YIELD TO ONCOMING TRAFFIC sign) shall be used in place of the TC-21 in the direction of the closed lane only.
- For low-volume roads, the TCP and sign for the lane that is not closed can be eliminated where visibility is 150m or more.

FIGURE A4.2 RIGHT LANE CLOSED — MULTI LANE ROAD

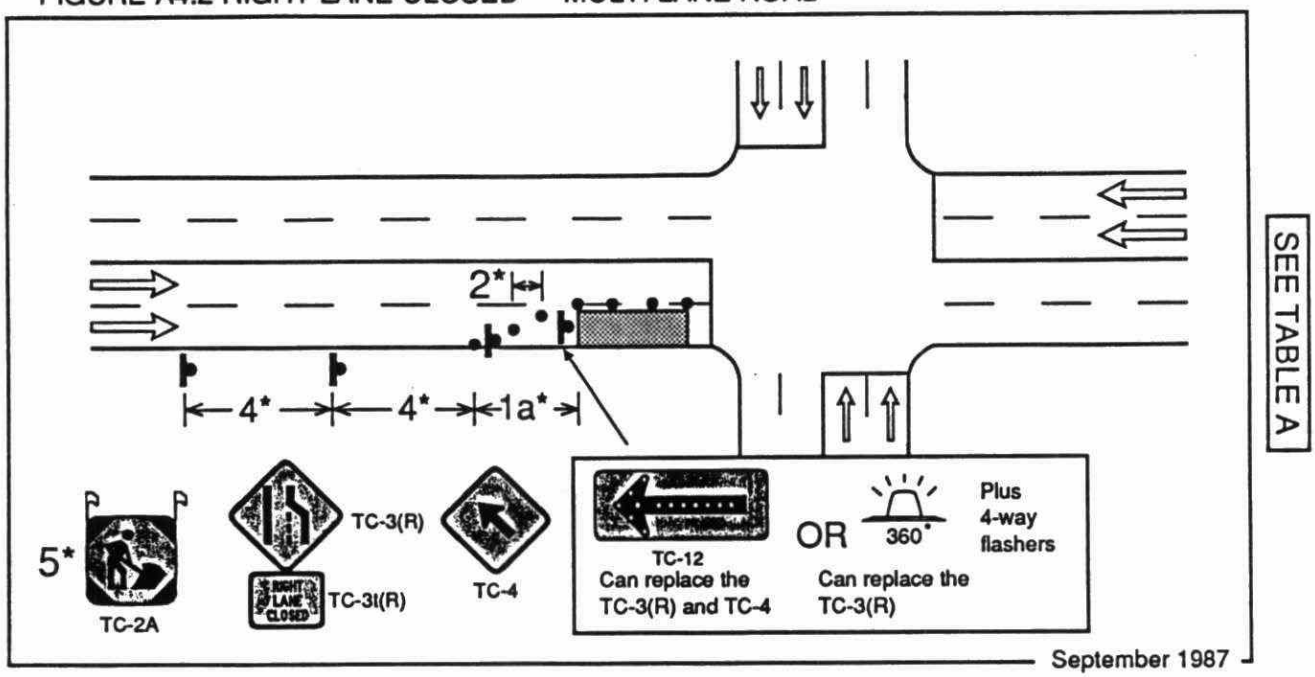


FIGURE A4.3 LEFT LANE CLOSED — MULTI LANE ROAD

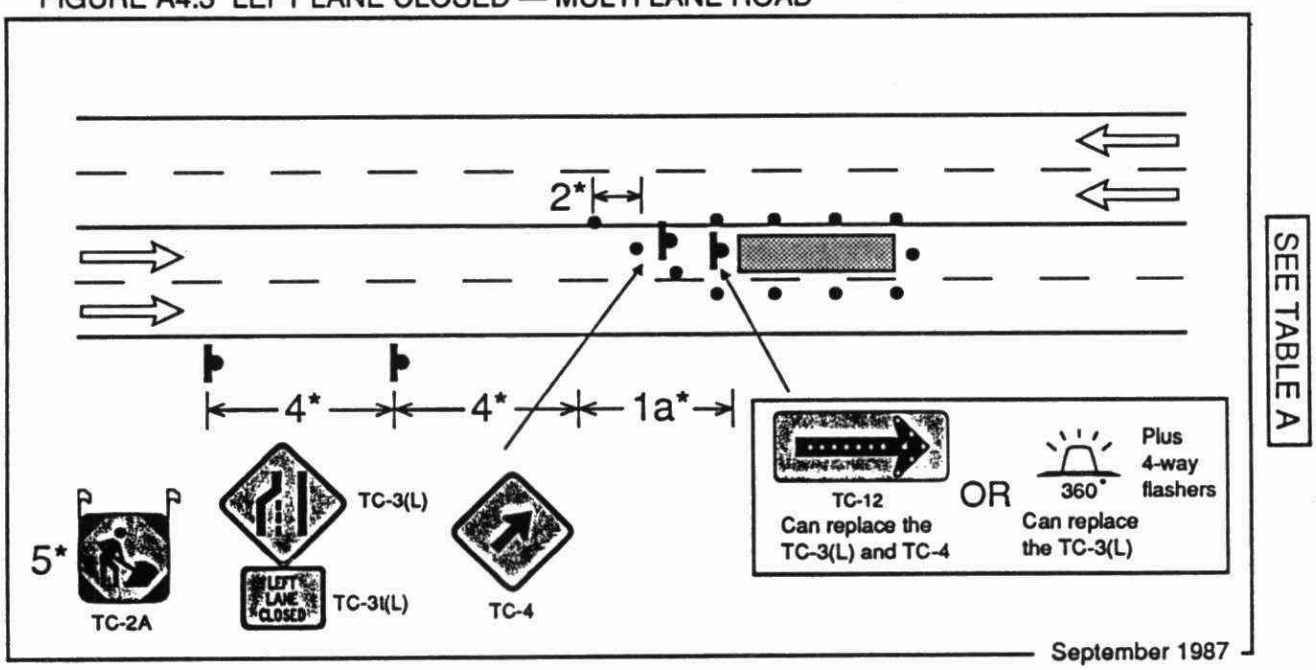
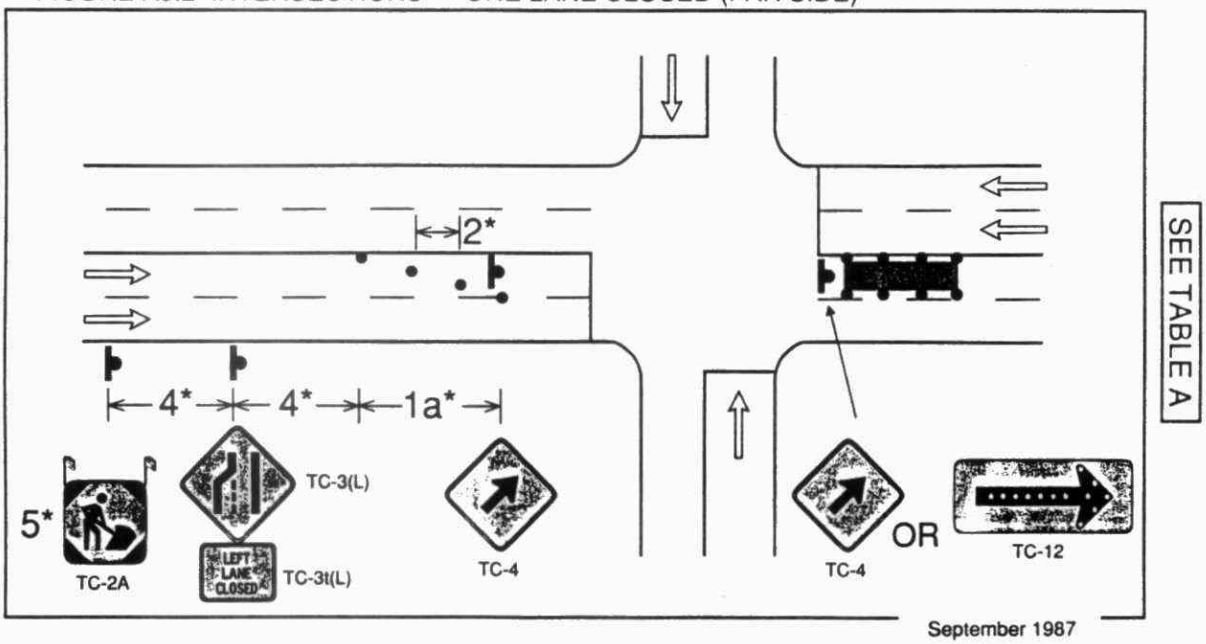
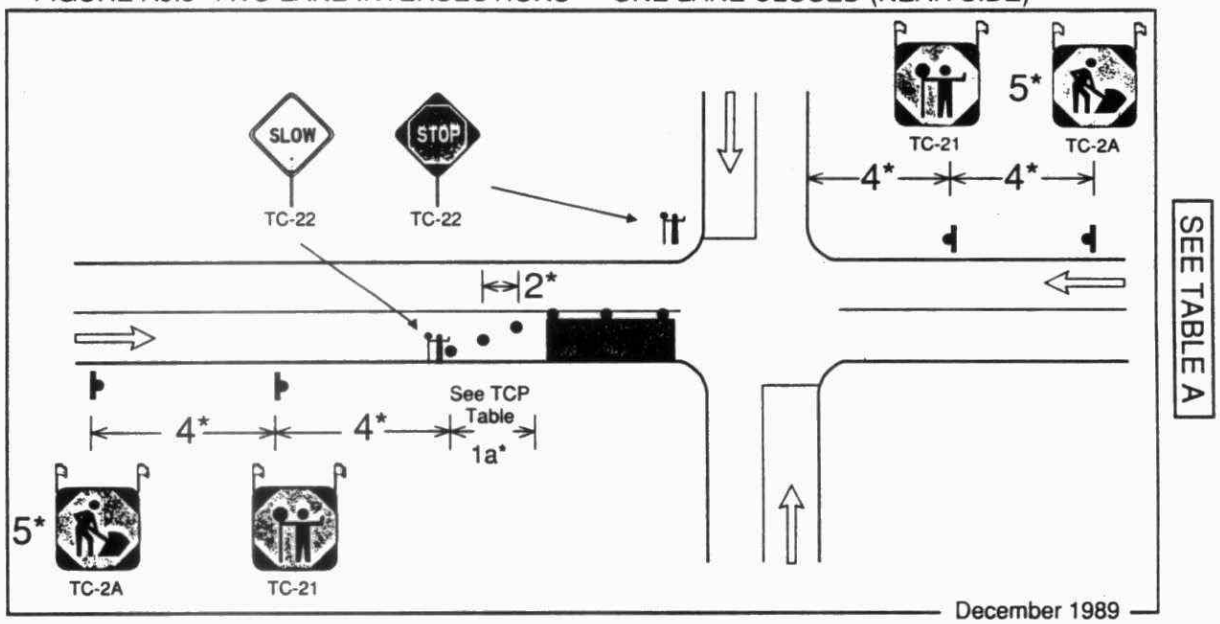


FIGURE A5.2 INTERSECTIONS — ONE LANE CLOSED (FAR SIDE)



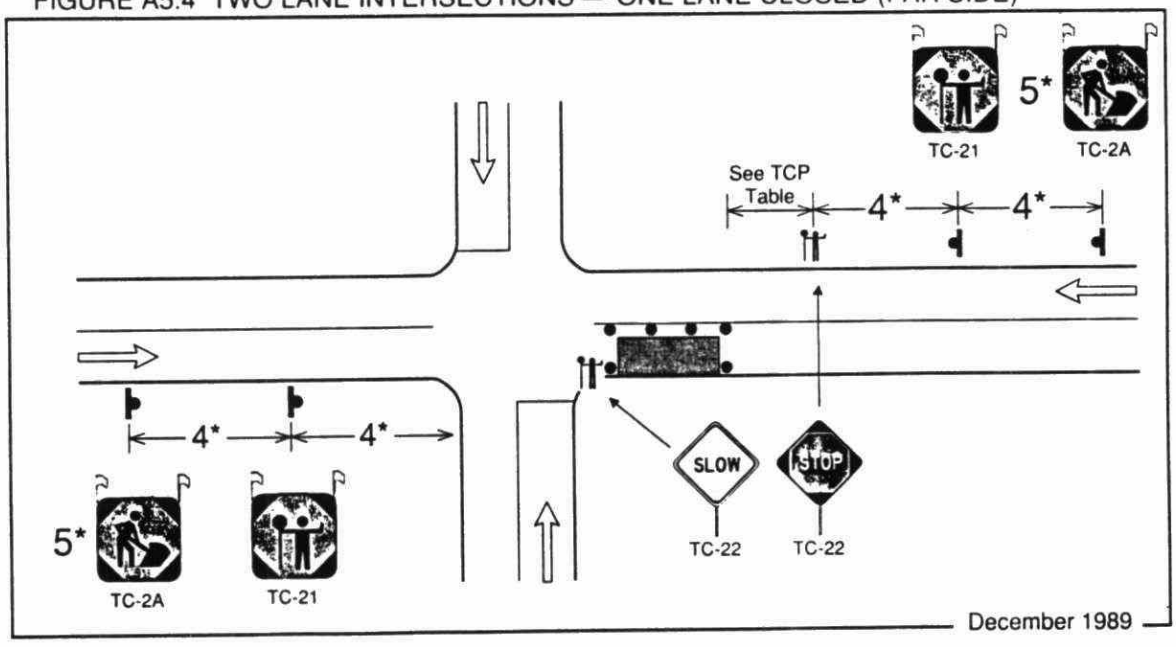
- It may be desirable to eliminate the lane closure in advance of the intersection if a heavy left turn volume may cause congestion.

FIGURE A5.3 TWO LANE INTERSECTIONS — ONE LANE CLOSED (NEAR SIDE)



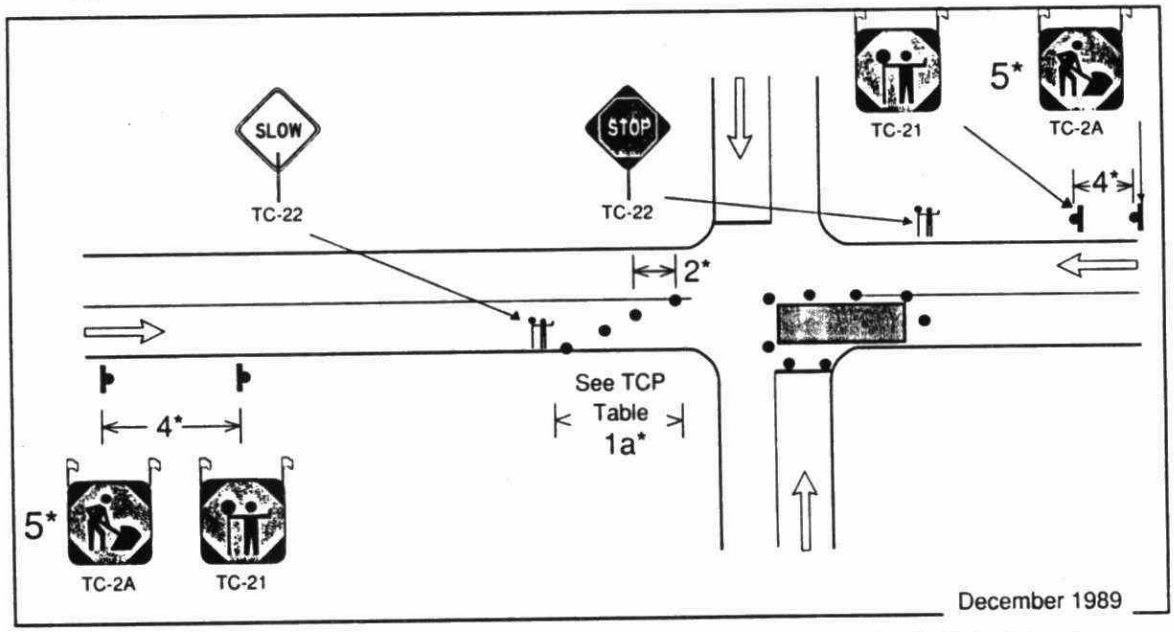
- When traffic volumes are high or when the intersection is signalized, consult the road authority to determine whether police assistance is required. Care should be taken by the TCP to co-ordinate with any intersection control such as traffic signals.

FIGURE A5.4 TWO LANE INTERSECTIONS — ONE LANE CLOSED (FAR SIDE)



- When traffic volumes are high or when the intersection is signalized, consult the road authority to determine whether police assistance is required. Care should be taken by the TCP to co-ordinate with any intersection control such as traffic signals.

FIGURE A5.5 TWO LANE ROADWAYS - WORK IN INTERSECTIONS



- When traffic volumes are high or when the intersection is signalized, consult the road authority to determine whether police assistance is required. Care must be taken by the TCP to co-ordinate with any intersection control such as traffic signals.

B-LONG DURATION WORK

Long duration work includes all construction and maintenance activities which require a separate work area greater than one day in duration. The following section presents the sign layouts for the most common types of long duration work operations (non-freeway conditions). They show the type, sequence and placement of the standard signing required for the various work situations.

Figure B1 **Advisory Signs**

Figure B2 **Roadside/Shoulder Operations**

- **B2.1** Shoulder Operations
- **B2.2** Roadside Work- Low Volume Unpaved Roads

Figure B3 **Lane Closure**

- **B3.1** Lane Closure-Two Lane Roads (TCP's)
- **B3.2** High Volume Two Lane Roads (portable Lane Control Signals)
- **B3.3** One Lane Closed-Multi-Lane Road
- **B3.4** Left Lane Closed-Multi-Land Road
- **B3.5** Two Lanes Closed-Four Lane Road
- **B3.6** Centre Two-Way Left Turn Lane Closed

Figure B4 **Intersections**

- **B4.1** Intersections-One Lane Closed (Near Side)
- **B4.2** Two Lane Intersections-One Lane Closed (Near Side)
- **B4.3** Intersections-One Lane Closed (Far Side)
- **B4.4** Two Lane Intersections-One Lane Closed (Far Side)
- **B4.5** Work in Intersections - (One Lane Closed)
- **B4.6** Intersections-Two Lanes Closed (Near Side)

Figure B5 **Detours**

- **B5.1** Roadside Diversion
- **B5.2** Route Detour

Figure B6 **Sidewalks**

- **B6.1** Sidewalk Closed
- **B6.2** Sidewalk Detours

Figure B7 **Narrow Lanes**

TABLE B — LONG DURATION WORK

	Regulatory Speed Limit	50 km/h or Less		60-70 km/h		80-90 km/h
*	Traffic Volume	Low Vol.	High Vol.	Low Vol.	High Vol.	Low or High Volume
1	Lane Closure Taper Length (m)	15-25	30-50	40-60	80-100	100-120
2	Max. Distance Between Markers (m)	6-8		8-10		10-12
3	Min. Tangent Between Tapers (m)	55		120		150
4	Distance Between Const. Signs (m)	40-50		100-150		150

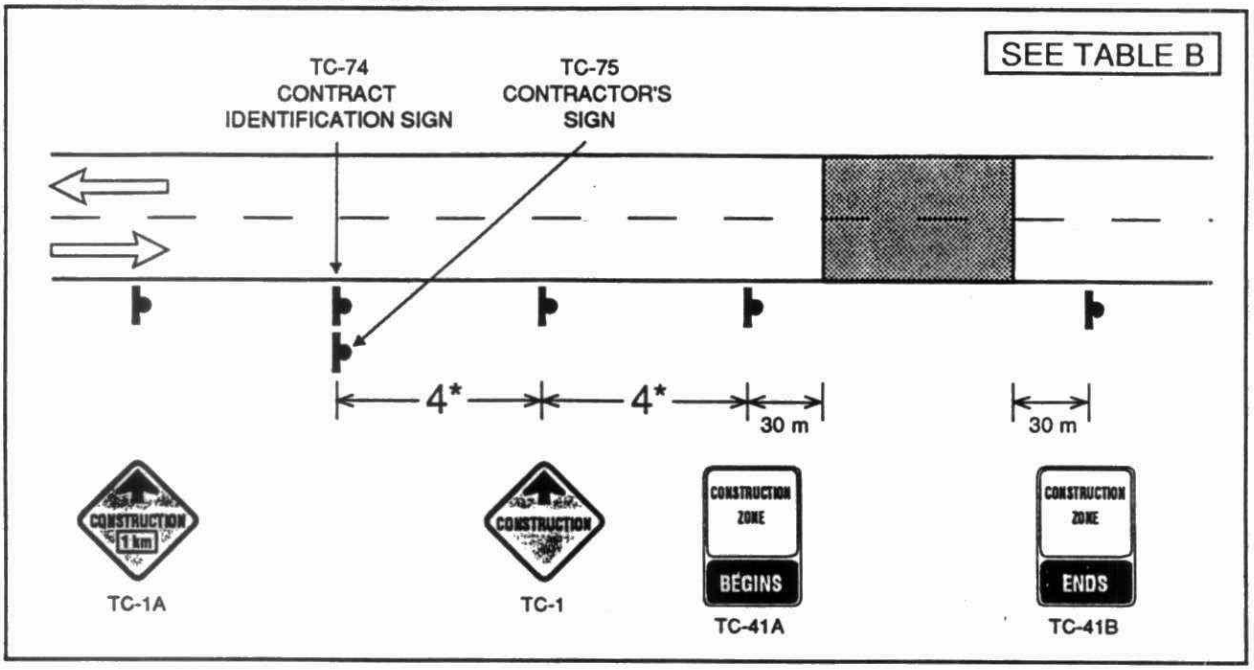
- Low volume is defined as an average daily traffic volume less than 3000 vehicles per day (combined traffic for both directions). This figure can be obtained from the local road authority or can be estimated by counting the number of vehicles passing the work site in 3 minutes and multiplying this figure by 300.

Example: 20 cars in 3 minutes X 300 = 6000 vehicles per day. (This would be a high volume road.)

NOTES:

- When the 85th percentile speed is known, it should be used instead of the posted regulatory speed limit.
- Cones may be used for daytime operations, however, construction markers or flexible drums shall be used for nighttime operations.
- All M.T.C. construction projects require the Advance Advisory signing shown in Figure B1.

FIGURE B1 ADVISORY SIGNS



- The TC-41A and TC-41B shall be used only if a construction zone is established
- The same signing is required in the opposite direction

FIGURE B2.1 SHOULDER OPERATIONS

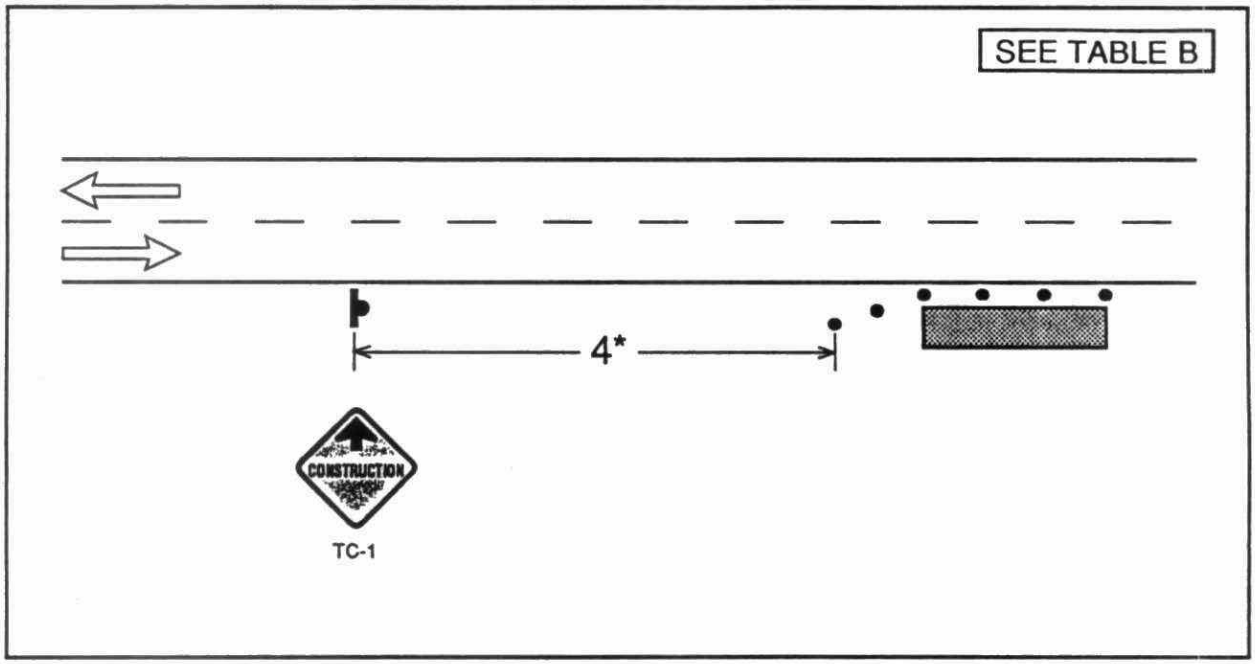
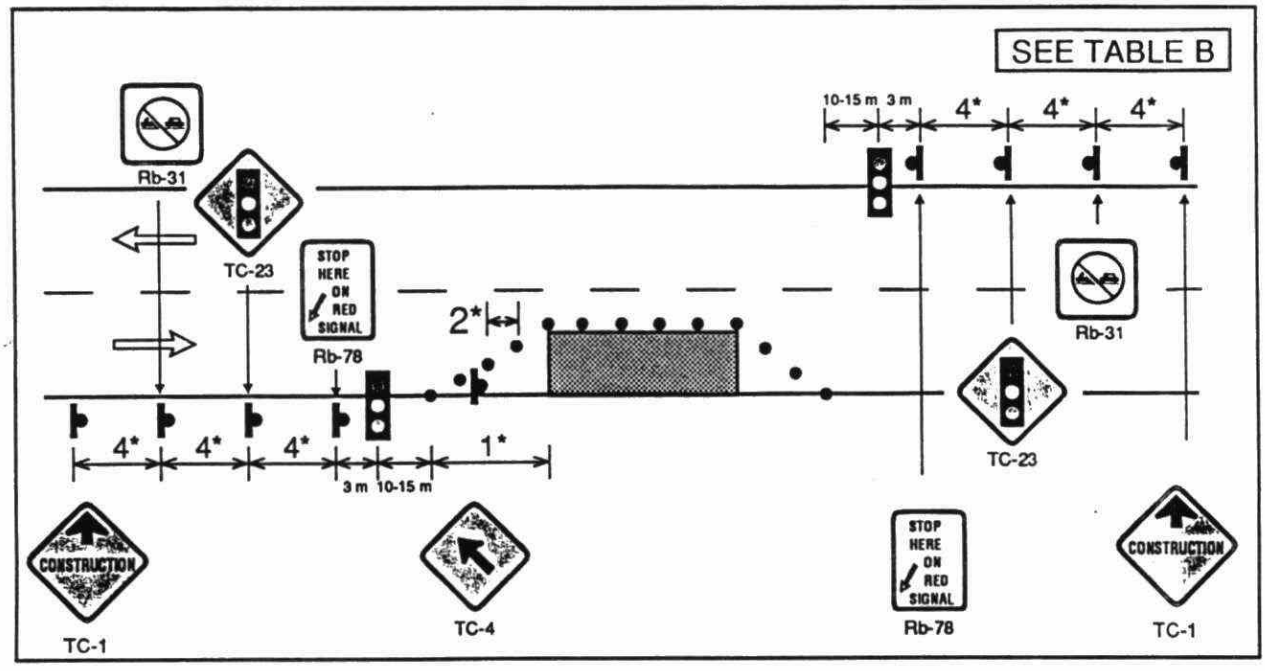


FIGURE B3.2 HIGH VOLUME TWO-LANE ROADS (PORTABLE LANE CONTROL SIGNALS)



- See section on Traffic Control to determine the appropriate timing of the portable lane control signals.
- Temporary traffic signals may be used to assist construction vehicles to cross roadways, however, approval must be obtained from the road authority having jurisdiction.

FIGURE B3.3 ONE LANE CLOSED — MULTI-LANE ROAD

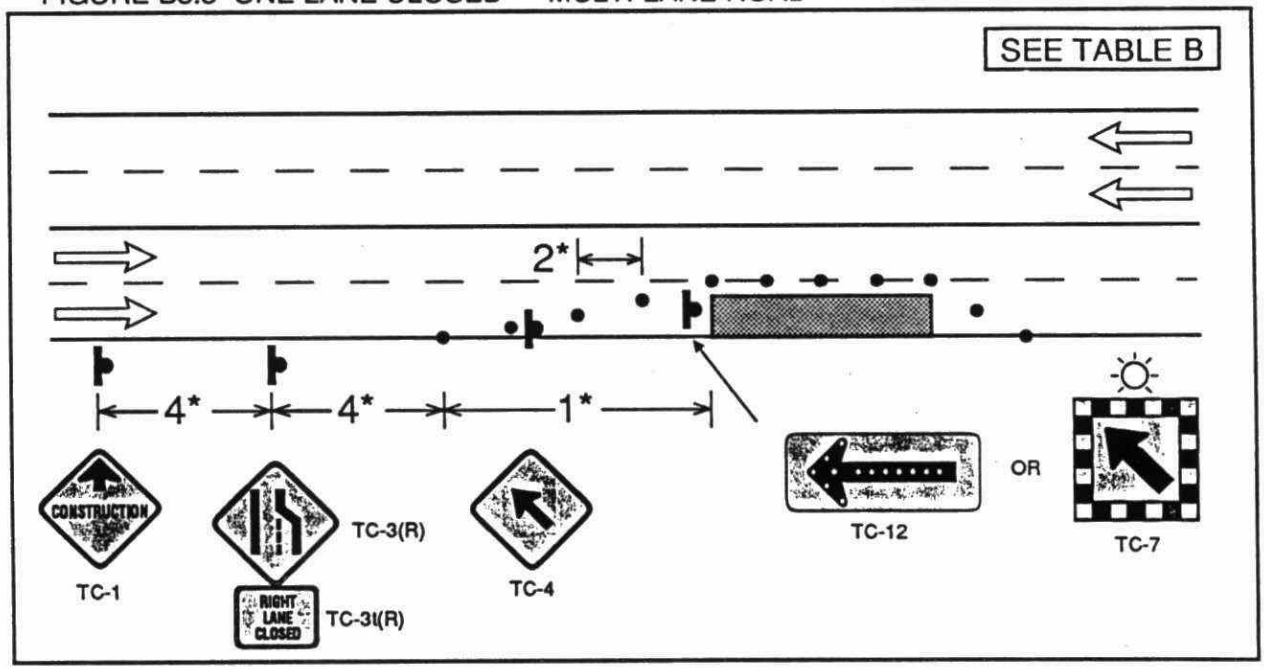


FIGURE B3.4 LEFT LANE CLOSED — MULTI-LANE ROAD

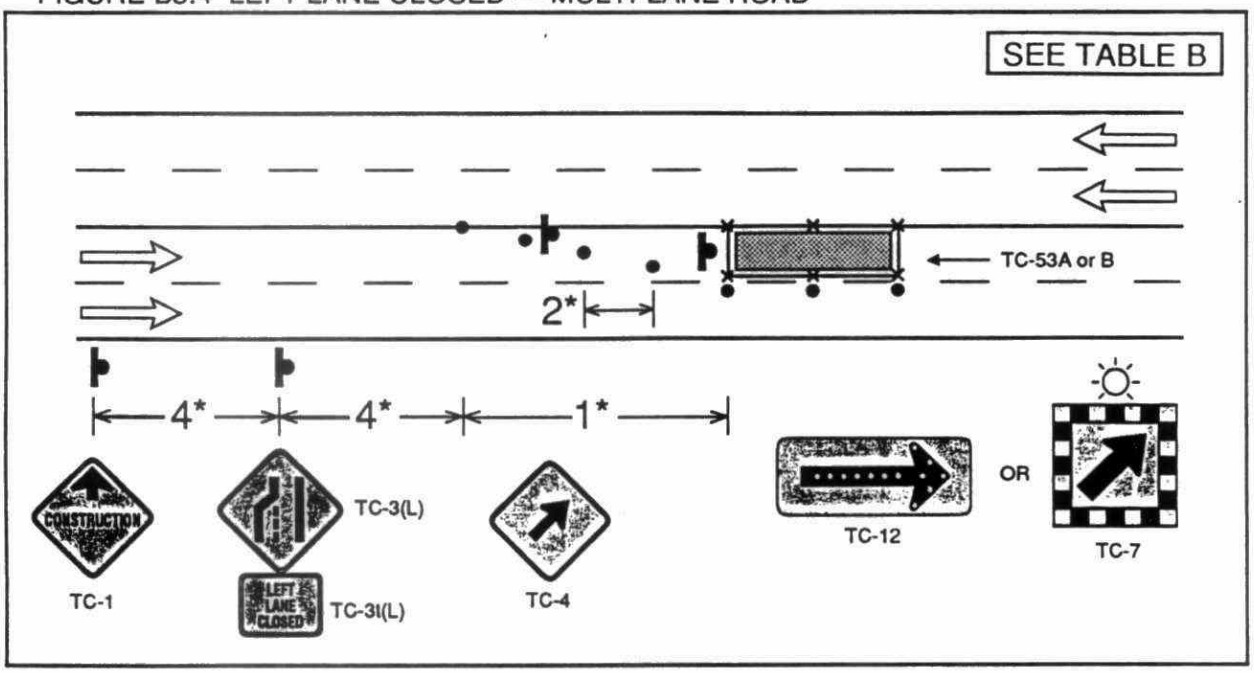
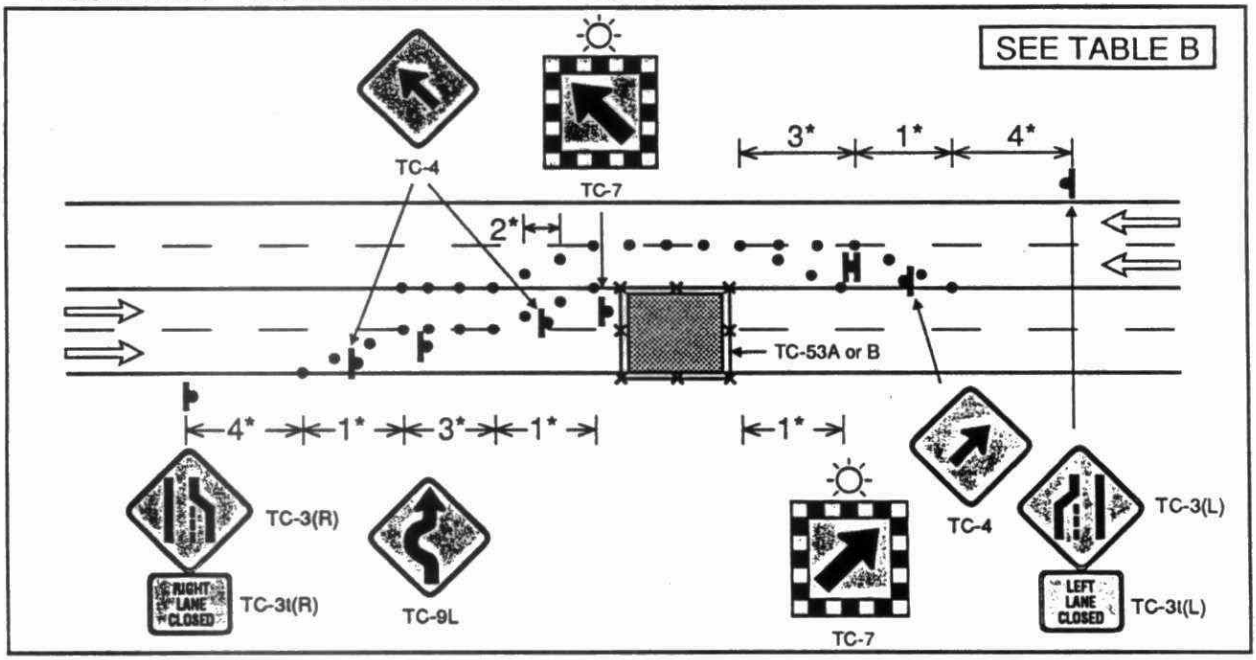
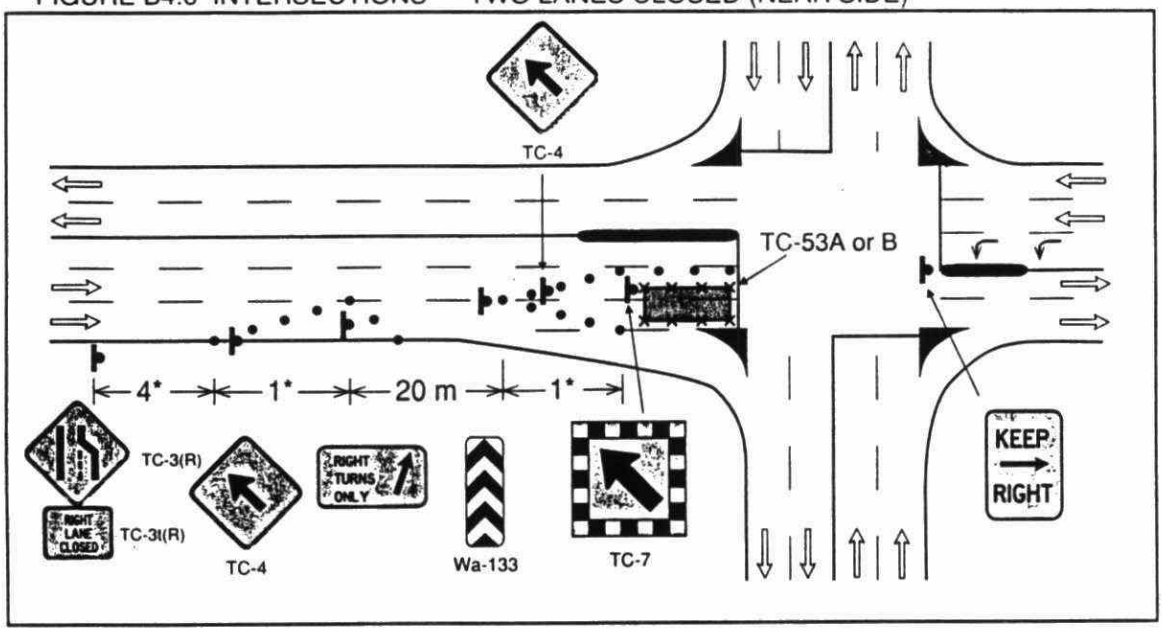


FIGURE B3.5 TWO LANES CLOSED — FOUR LANE ROAD



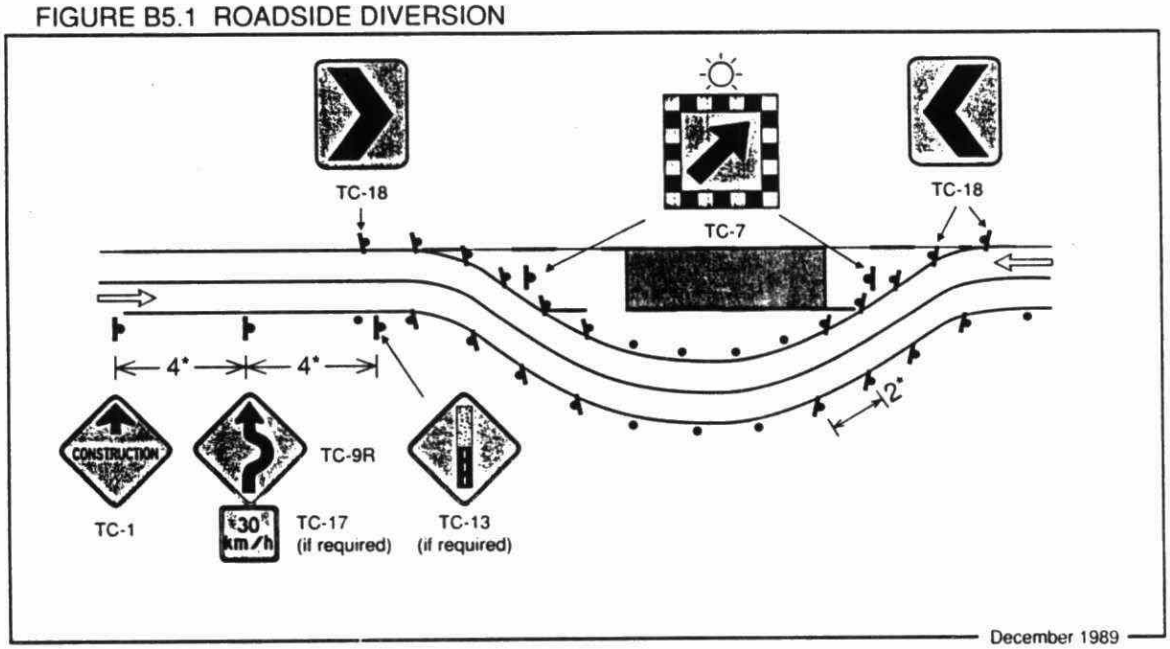
- For high volume roads, the use of New Jersey Type Concrete Barriers should be considered to separate opposing traffic.
- TC-53A or B barriers should be used to protect the worksite.

FIGURE B4.6 INTERSECTIONS — TWO LANES CLOSED (NEAR SIDE)



- It may be necessary to prohibit certain turning movements
- The KEEP RIGHT sign is to be used only if an Rb-25 (Keep Right) is not already in place

FIGURE B5.1 ROADSIDE DIVERSION



- If the diversion is paved, temporary pavement markings are required, including edgelines
- The same signing is required for opposing traffic

SUBJECT:

CONSTRUCTION OF SEWERS WATERMAINS AND ASSOCIATED FACILITIES

TOPIC: 8

SANITARY AND STORM SEWER DESIGN

OBJECTIVES:

The trainee will be able to:

1. Understand the basic concepts in the design of sewage collection systems.
2. Differentiate between sanitary and storm sewage systems.
3. Realize the value of proper construction procedures.

SANITARY AND STORM SEWER DESIGN

GENERAL ASPECTS OF SANITARY SEWERS

Whether designing a small section of sewer to serve a newly developed portion of a municipality or an entire system there are certain overall considerations which must be examined. The designed must forecast what the future of the particular area will be. The forecast will be more accurate if an official plan exists upon which the designer can base future land use predictions. Some of the factors affecting the design are listed below.

Area to be Serviced

The area under study may be limited by topography, political boundaries or by economic factors. When providing sewers for an area one should look beyond the boundaries set down for the immediate study and make provision, if necessary, for the inclusion of adjacent areas at a later date.

If an official plan is available then the selection of the areas within the municipality has already been outlook and the need for sewers and other services has had some consideration. Land use patterns are predictable and scattered development may be avoided by proper zoning by-laws.

Land Use

The actual nature of the development in the area may be dictated by political and economic considerations; however, in modern urban areas planning has helped reduce scattered and chaotic growth. All municipalities contain residential, commercial and industrial areas. One of the problems in the design of the system is to predict the location, nature and extent of each of these areas.

It is obvious that the size of sewer required will be quite different on a suburban residential street than on a road containing high-rise apartment buildings. Commercial areas may cause large peaks in water usage and resultant sewage flow.

The land use pattern must be carefully studied to give the designed some idea of the quantity, and nature of sewage that will be discharged now, in the future, in a local area, and in the entire municipality.

Population Prediction

Relative growth of an area or municipality may be dictated by factors such as:

1. Proximity to large urban areas
2. Number and type of industries present
3. Government policy
4. Economic situation of the region

The effect of Metropolitan Toronto on the surrounding municipalities need not be emphasized. The number and type of industries present will affect the population growth of a town in that the industry already established will have necessitated a certain type of service being made available to the industry, thus perhaps allowing additional industry to be served at little inconvenience. The Provincial Government may have declared a certain area to be "depressed". In such an area tax relief may be given to industries which would settle there, thus making this area more attractive than another area, all other things being equal. The municipality may be very aggressive in advertising for new industries and convincing representatives of industry to locate in their municipality. The economic prosperity of a region will very much influence the ability of that area to progress.

All of the above factors will influence the rate at which the population increases and hence the need for municipal services such as sewers.

DETAILED ANALYSIS OF THE AREA

After an analysis of the general aspects of design it becomes necessary to closely examine the system and the various factors influencing design.

Analysis of the Area

The topography of the area - soil conditions, natural boundaries, ground elevations, ground cover - will determine the physical layout of the collection system. Large trunk sewers can be designed to follow natural valleys and drainage areas. Collector sewers will generally be laid on road allowances and, in most instances, in the centre of the road allowance to permit similarity of design for service connections.

Ground Cover

In all sewer alignments consideration must be given to the nature of the area to be disturbed. In valleys and rural areas soil erosion, excessive clearing, river silting, improper dumping should be controlled. In built-up areas, noise control, traffic control, emergency vehicle access, trench settlement, protection of services are important design considerations.

It is good design to leave an area in a condition as close to the original as possible. Photographs of a proposed route should be taken immediately before construction commences.

Soil Conditions

Knowledge of soil conditions of the area is essential. Soil conditions may render the provision of sewer services in some areas uneconomical. Poor soil conditions will affect the cost estimate as well as influence construction methods and materials. If a large quantity of rock has to be excavated then obviously the cost for the system will be must greater than the cost for sandy soil conditions. Conditions such as rock or muskeg may require that certain services be provided in a common trench to prevent excessive excavation and/or use of imported backfill material.

A high water table in a pervious soil may cause excessive infiltration in a sewer and require specific bedding considerations.

Population Prediction Methods

Before attempting to calculate the anticipated flow it is necessary to estimate the future population of the area. The interpretation of past records may be used in conjunction with factors mentioned previously under General Aspects, sub-section 3. Several methods may be used to predict future populations from past records, including:

1. graphical extension of past records into the future;
2. comparison with similar cities of somewhat greater population;
3. mathematical analysis.

Method 1. above is one of the simplest and most frequently used methods of predicting future population.

At this point it is important to make the distinction between projected population and serviced population. It is probable that all the population cannot be serviced for economical or physical reasons. For example, it may be uneconomical or physical reasons. For example, it may be uneconomical to service pumping station or by lowering the sewer. Thus, provided that the septic tanks are working satisfactorily, it may be worthwhile to not service these houses. The fact to stress is that only the serviced population will contribute to sewage flows.

Distribution of Population

In order to effectively design a system, it is necessary to know not only the future population of the community, but also the densities throughout the area served by the sewer system. The average density may be obtained by taking the total predicted population and dividing this number by the habitable area of the community.

It is essential for a modern community to have an official plan of development. Knowing the nature of the development that will take place in various sections of the community, the designer can assign probable saturation densities as determined from previous experience. In this manner an estimate can be made of the population of the tributary area of each sewer involved. The saturation density of all the sectors of the community will undoubtedly exceed the predicted population of the community as it does not usually happen that the community is fully saturated with people at the end of the design period.

FLOW PREDICTION

Sewage Flows

With a reasonably good knowledge of soil conditions, topography, area, land use and populations, the problem of estimating sewage flows may be attempted. Waste flows from the following sources may be summed up to give the total flow:

a) Domestic
b) Industrial
c) Commercial
d) Institutional
e) Infiltration of Ground Water
f) Storm Water

Values for the first four may be obtained by analyses of the problem and application of information gained through past experience. The effect of infiltration is minimized by good construction; the effect of storm water is minimized by eliminating combined sewers, roof drains and footing drains where possible.

Common Expedient

Due to the obvious difficulty in obtaining predictions, the designer frequently finds it a satisfactory expedient to simply assign a per capita flow which is meant to include all contributions. Such a value may be obtained by examining past water consumptions records or the per capita sewage flows from similar municipalities. A common figure used in design for water consumption and sewage flows is 500 litre/capita/day average flow.

Unless circumstances require that a higher or lower figure be used, the 500 litre/capita/day is used for the design.

ACTUAL DESIGN OF SEWERS AND APPURTENANCES

With the information now available, the designer can proceed to select the actual sewer sizes. Items which must be considered now are design period, design flow and hydraulic factors.

Design Period

The design period is that period during which the facilities are expected to be adequate. If a 20 year design period is chosen, then the designer expects that in the 21st year the sewer will be inadequate and will have to be replaced with a larger one. The normal design period is 20 years.

Design Flows

The 500 litre/capita/day figure previously mentioned is an average figure for a 24-hour day. Since people obviously do not use water at a constant rate over 24 hours, it is necessary to predict the peak flows to be expected during the day. These peak flows occur in the morning between 6 to 9 a.m., at noon, and again between 4 to 7 p.m.

In order to obtain these peak flows a "Peaking Factor" is used. The average daily flow (i.e. the 500 litre/capita/day) is multiplied by a factor of between 2.4 to 5 to obtain the maximum flows to be expected. The following formula may be used to determine the peaking factor:

$$\text{Peaking Factor} = \frac{\text{max expected flow}}{\text{average daily flow}}$$

$$M = 1 + \frac{14}{4 + \sqrt{P}}$$

$$P = \text{tributary population in 1,000's}$$

A table of peaking factors is given below for populations up to 10,000.

P = 1	VP = 1.00	F = 3.80
2	1.415	3.59
3	1.73	3.45
4	2.00	3.34
5	2.24	3.24
6	2.45	3.17
7	2.65	3.11
8	2.83	3.04
9	3.00	3.00
10	3.16	2.95

Reduced peaking factors are sometimes employed for industrial contributions, since industrial peak can be normally predicted with greater certainly than can domestic peaks.

Infiltration

When designing sanitary collector sewer systems, an allowance should be made for the leakage of groundwater into the sewers and building sewer connections (infiltration) and for extraneous water entering the sewers from sources such as manhole covers. The infiltration rate is usually determined using the drainage area and typically ranges from 0.25 L/S/ha to 0.30 L/S/ha.

Grade

The grade of the sewer will be influenced considerably by the ground surface. That is, in order to provide a fairly uniform depth of sewer, the slope of the sewer must generally follow the existing slope of the ground. Sewer grades must provide self-scouring velocity at full flow conditions; that is, the minimum velocity at full flow conditions must be at least equal to 0.6m per second, the velocity at which settling out of solids will be prevented. A partial list of minimum grades is presented below:

Diameter of Sewer (mm)	Theoretical Minimum Grade (m/1000 m) @ n = 0.013	Capacity (L/sec)
200	4.0	20.7
250	2.8	31.5
300	2.2	45.4
350	1.8	61.9
400	1.5	80.7
450	1.2	98.8
500	1.1	125.2
600	0.8	173.6
700	0.67	240.1
800	0.58	318.5
900	0.46	388.3

In general, the cost of sewer construction will increase in proportion to the depth of the trench, modified by the condition of the soil, depth of rock, location of ground water, need for shoring and other sub-surface factors. Sewer depths greater than 4.5 metres are considered inadvisable and a minimum depth of cover of 1.8 metres is usually acceptable, but the sewer must be deep enough to permit basement drainage. In some cases, it may be advisable to provide a few houses with sump pumps to pump the basement drainage up to the sewer rather than lowering the whole sewer significantly and thus greatly increasing the cost of the sewer at a lower depth. There have been specialized cases where a complete system of shallow sewers have been installed to keep the cost of the sewers from being prohibitive with sump pumps in almost every house to provide basement drainage.

Size of Sewer

After the designer has determined the sewage flows and grade limitations, he may proceed to select the appropriate sewer size. Formulae relating capacity, and hydraulic gradient, co-efficient of friction, and pipe sizes are available.

To improve maintenance and prevent blockages, a minimum sewer size of 200 mm is required in the Province of Ontario. In the average community, most laterals need only be of the minimum size of 100 mm.

PUMPING STATIONs

Requirements for a Pumping Stations

In general, sewage pumping stations may be required under the following conditions:

1. Where topography is such that good sewer grades by gravity flow are not possible without excessive depth and with consequent high construction costs. The sewer grade is carried as low as is practical, and then the sewage is lifted by a sewage pumping station in order that the downstream sewer may be placed closer to the ground surface, thus reducing construction costs. This case is encountered where the topography is very flat.

2. Where the hydraulic gradient is such that there is insufficient head for gravity flow through a treatment plant.

3. Where it is necessary to boost the sewage over a ridge through a forcemain, to a point from which it will flow by gravity.

It is good practice to equip the pumping station with an overflow in case of forcemain, mechanical, or electrical failure. Standby power must be provided to prevent overflows. The overflow of sewage into a recreational stream or upstream of a water intake or onto streets or into basements by sewer back-ups might be a serious health hazard. Standby power may be provided in the form of either a diesel generator set or a direct-drive diesel engine. If standby power is provided at a sewage pumping station, it must be located in an above-ground structure.

Forcemain

The forcemain from the station must be selected to minimize friction losses but at the same time to provide a minimum scouring velocity of 0.75 m/sec. in the pipe to prevent deposition of solids. The maximum velocity in a forcemain is usually limited to a maximum of 3.0 m/sec. since velocities in excess of this figure will cause very severe head losses and resulting high heads required for the sewage pumps and high power costs.

Inverted Syphon

The purpose of the inverted syphon or depressed sewer is to carry the flow under an obstruction such as a river or depressed highway and to gain as much elevation as possible after the obstruction is passed.

An inverted syphon is an arrangement whereby the sewage flows down the upstream side of the syphon and is forced up the downstream side by the pressure of the inflowing sewage. Syphons generally consist of two or more pipes (barrels) with the smallest laid in the lowest position and designed to take minimum flows; higher flows spill into the second pipe during peak flow conditions. Velocities in the range of 0.9 m/sec. should be achieved to prevent blocking of the syphon. Control gates may be removed from service. Drains and clean-outs should also be provided to allow maintenance if necessary.

Manholes

The purpose of manholes is to permit access for inspection and cleaning of sewers, merging of flows, and ventilation. Manholes are located at the junctions of sewers and at the changes in alignment or grade and diameter. Street intersections are common locations for manholes, as junctions frequently occur there. It is common practice to have a manhole at the upstream end of a sewer for convenience in flushing and cleaning, and in the event of a future extension of the sewer. The recommended distances between manholes on a straight run is as follows:

- 90m for sewers 400mm diameter or less
- 120m for sewers 450 - 800m diameter
- for sewers greater than 800mm diameter the spacing may be increased especially where sewers are big enough where a man can actually walk through. The spacing, however, also depends on the cleaning equipment which the municipality has. If modern and efficient cleaning equipment is available in the municipality then the manhole spacing may be greater than a similar municipality which might have poor cleaning equipment.

Manhole spacing is affected by grades available. If flat grades are necessary then manholes should not be spaced too far apart since blockages are more likely than if good sewer grades are possible. At the other extreme, sewers going down steep hills may require "drop" manholes, closely spaced, to dissipate energy.

House Service Connections

The general purpose of constructing a sewer is to allow connections to be made to it from abutting properties. The flow characteristics in the sewer can be affected by the manner in which connections are allowed to be made and the number and relative size of connections made. Wherever possible, factory-made wye junctions should be installed at the time of original construction and the designer should make every effort to anticipate the number of connections required. In excessively deep sewers, it is quite important that risers be installed at the time of construction and brought to a usable depth below finished grade.

Nowadays storm and sanitary systems are usually separate and it is imperative that a method be devised to made sure that private drain contractors know the difference between the storm and sanitary connection lines and are made to realize the importance of connecting properly. In spite of regulations and inspection, there is an alarming amount of storm water entering into the sanitary sewers. No amount of careful design can affect poor control over use of connections. On well-inspected sewer construction, sewers are adequately tight against infiltration, yet each new sanitary sewer, after private connections are made, has a marked increase in flow rates during and immediately after a rain storm. Close supervision of construction of house service connections, is therefore, an important part of the Inspector's work.

Example: Given - A village is situated as shown in Figure 8-1.

After analyzing future trends it is found that this small community is going to develop in 20 years with a mixture of residential, commercial and small industrial areas to approximately 1500 persons on the east side of the river and 500 persons on the west side. The ideal site for the sewage treatment plant is shown south of the village on the west side of the river.

The designer has examined past records and determined that the average water consumption has been 300 litres per capita per day. He has further decided to use a 20-year design period for his treatment plant. Since the community is small and of balanced composition, he has determined that he can conservatively use a per capita sewage contribution of 375 litres/capita/day - this will account for the current trend toward increased water usage.

An analysis of the topography shows that the village can be separated into two drainage areas with collector sewers on each side of the river. A sewage pumping station will be required at the east side of the pumping station will be required at the east side of the river; it will discharge into a manhole on the west side; from there the sewage will flow by gravity to the plant.

An examination of the drainage area shows that all laterals need be only 200mm diameter - even for the 50-year life of the sewer. The design of the collector sewers may be computer as follows:

East Collector

Basic design data: east side	-	2000 persons - 50 year population (saturation)
Average per capita contribution	-	375 Lpcd
Maximum expected flow	-	2000 x 375 x peaking factor
Peaking factor	=	$1 + \frac{14}{4 + \sqrt{P}}$
	=	$1 + \frac{14}{4 + \sqrt{2}}$
	=	3.59
maximum expected flow	=	2000 x 375 x 3.59
	=	2,692,500
Infiltration, assume	=	159,250 LPD
Design Flow		2,851,750 LPD

Using the table on Page 8-8, a 300mm sewer at minimum grade could be used.

West Collector

From similar calculations the designer finds that at 200mm diameter sewer will suffice.

Pump Station

10-year population	-	1200 persons
20-year population	-	1500 persons
10-year average flow	-	1200 x 375
	=	450,000 litres + infiltration
20-year average flow	-	1500 x 375
	=	562,500 litre/day + infiltration

Since the two flows are reasonably close, the design will be for the 20-year flow, for which the peaking factor is 3.76.

Maximum expected flow	-	1500 x 375 x 3.76
	=	2,115,000 litres/day
Infiltration, ass	=	160,000 litres/day
Total	=	2,275,000 litres/day
	=	1580 litres/min

In order to provide a velocity of 0.76 m/sec. a 200mm diameter forcemain is required.

It will be necessary to choose a wet well/dry well type of pumping station with prefabricated dry well, for economical reasons. Two 1580 L/min. pumps will be provided in order to provide a standby pump in case of a mechanical failure.

Trunk Sewer

The size of the trunk sewer from the manhole (to which the pumping station discharges) to the sewage treatment plant is computed by using the 50-year flow, and the minimum slope required to maintain the 0.6 m/sec. velocity.

Since the trunk sewer must be designed on the 50-year flow and in the absence of any other information, assume that the west side of the town grows in the same proportion as the east side of the town. Thus, the 50-year total population of the town may be computer by the following ratio:

$$\frac{\text{50-year population (east side)}}{\text{20-year population (east side)}} = \frac{X}{\text{20-year population (whole town)}}$$

where x = 50-year population of the whole town

$$x = \frac{2000 \times 2000}{1500} = 2666$$

for a population of 2666, the peaking factor = 3.5

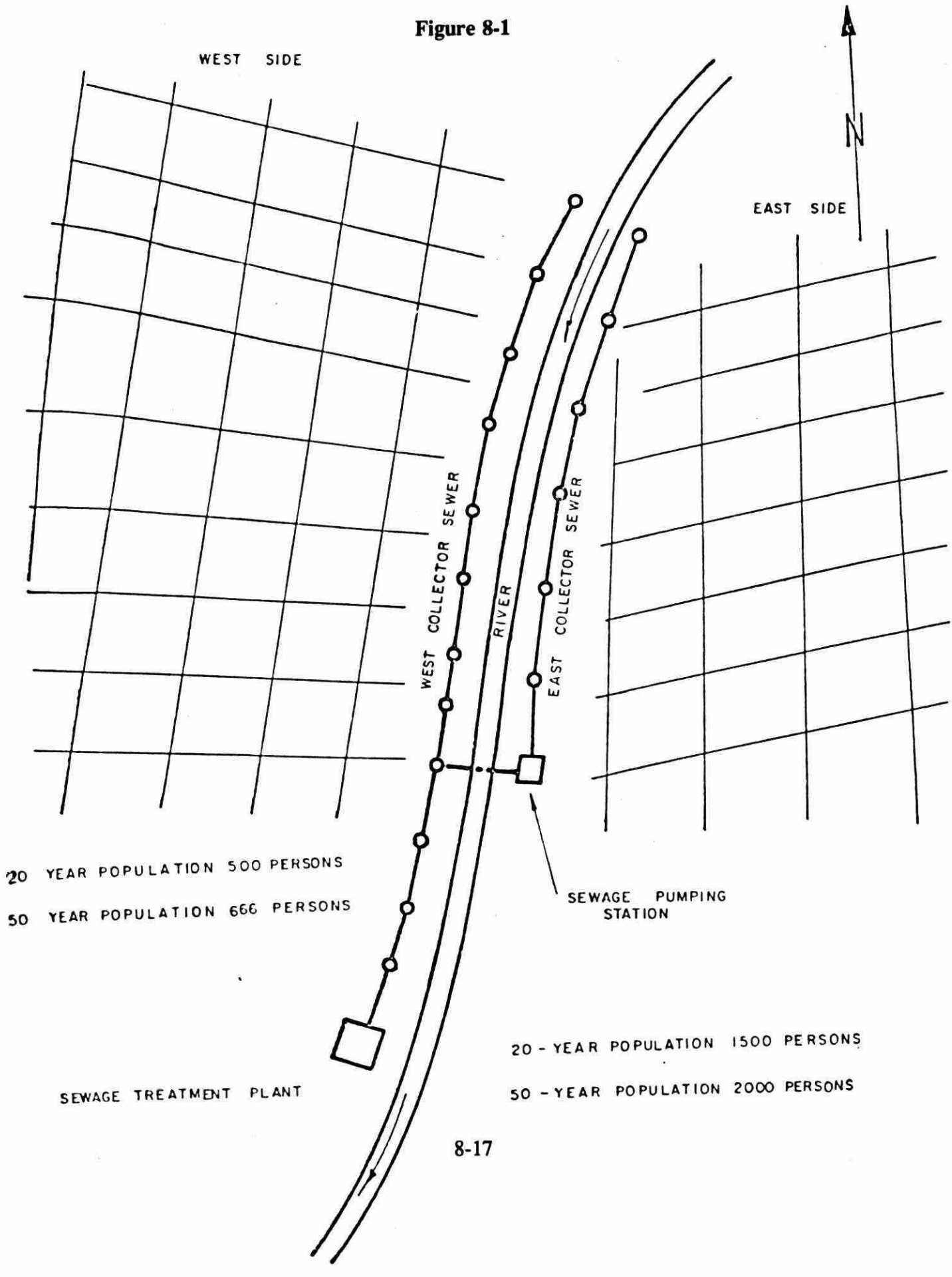
maximum expected flow = 2666 x 375 x 3.5 = 3,499,125 litres/day

infiltration, assume = 11,400 litres/day

Required capacity = 3,513,125 litres/day
= 40.5 litres/sec.

From the table on Page 8-8, it is found that a 300mm diameter sewer will suffice.

Figure 8-1


WEST SIDE
EAST SIDE
N
WEST COLLECTOR SEWER
RIVER
EAST COLLECTOR SEWER
20 YEAR POPULATION 500 PERSONS
50 YEAR POPULATION 666 PERSONS
SEWAGE PUMPING STATION
20 - YEAR POPULATION 1500 PERSONS
50 - YEAR POPULATION 2000 PERSONS
SEWAGE TREATMENT PLANT

The purpose of a storm sewer system is to remove storm water rapidly and effectively and direct this water to a suitable outlet, usually a creek, river or lake.

Municipal engineers involved in the design of such sewers must be aware of both the hydrologic elements and the hydraulic element which affects the design.

The hydrologic phase of urban design involves the determination of magnitude, distribution and timing of various run-off events. Run-off is the part of rainfall not lost by infiltration into the soil or left in the ground depressions and on vegetated surfaces to evaporate. A number of the factors that affect run-off are: climate; duration and distribution of rainfall; initial soil moisture conditions; soil type; the shape; size and slope orientation of the rainfall area; and the land use characteristics of the general drainage area.

The hydraulic elements include the determination of the sizes, depths, shapes and materials of the proposed sewer construction.

The Design Flow

The difficulty with storm sewer design is deciding on the size of storm which will be used for the peak design flow. A well designed drainage system provides maximum protection at minimum costs. An economic balance is necessary between the cost of pipes, conduits and structures and the direct and indirect cost of public damage to property and inconvenience to the public over a long period of years. In areas where the effect of minor flooding would not be damaging, the system may handle ordinary storms but be insufficient to handle infrequent or more intense storms. On the other hand, in valuable city and highly built-up areas, where even minor flooding could be disastrous and very costly, the system must be designed to handle the more severe storms.

Drainage systems are designed to handle storms having frequencies of two years to ten years or more, depending on the value of the property. In commercial areas and for floodwall structures this frequency is often as high as fifty years. For purposes of design, the rainfall frequency is usually established by each municipality. Design frequency, as the words are used here, is the frequency with which a given event is equally or exceeded on the average, once in a period of years. Thus a ten year frequency event would be expected to be equalled or exceeded ten times in 100 years.

Peak flows result from excess surface run-off volumes. The conditions which may generate these excesses are intense storms, snow melt, or snow melt combined with rainfall.

The design flow will generally be defined as the maximum flow accumulated during and after the storm and the conduits or structures will be designed to accommodate this flow. The methods of calculating this design flow are discussed in the next section, but it is necessary to describe the actual flow happenings during a storm first.

A drainage area can be likened to a river system with flow coming from each small stream to form a larger stream and eventually a main river. If the area is small it may be covered entirely by the storm and the peak flow will be attained within the rainfall area at a specific point in the system after a certain time of concentration of the storm and this peak will pass down the system like a wave if the storm duration is short or as a large continuous mass if the storm duration is long. If the drainage area is large the peak may be found near the outfall of the system although the storm is some distance up the system.

In most instances, the design flow can become too large to allow realistic or economical sizing of structures and conduits.

It is now common practice to provide storage areas throughout a drainage area to control the peak wave flow. Instead of trying to design a system for the maximum flow of a ten year storm it is much better to control the inflow of the storm water to the system and design conduit and structures for a lesser storm peak but handle the larger storm by creating a lengthy peak flow condition in the system as the storage areas empty some time after the storm is over. Such areas can be swales, roofs of plazas, parking lots, fields, man-made lagoons, etc.

Procedures for Estimating Run-off

Procedures used in estimating run-off magnitude and frequency have greatly improved in the last two or three decades and can be generally categorized as: (1) empirical approaches; (2) statistical or probability methods and (3) methods relating rainfall to run-off.

Of the three approaches mentioned, the third one, i.e. to relate rainfall to run-off is the most widely accepted method and can be applied to both the "unit-hydrograph" method and the "rational method". The rational method is useful for first estimates but has been shown to be less consistent than the computerized mathematical models now generally in use. Nevertheless, this method will be discussed briefly as it has formed the basis of much storm sewer design to the present day.

The Rational Method - includes Hydrologic and Hydraulic Elements

This method, first introduced in 1889, relates rainfall to run-off by the equation:

$$Q = 2.78\ Air$$

Where Q = peak run-off in L/sec.
R = run-off coefficient
i = average rainfall intensity (mm/hr) for a period equal to the time of concentration
A = drainage area in Hectares

Two basic assumptions which must apply when using this method are: (1) the maximum run-off rate for any design location is a function of the average rate of rainfall during the time of concentration and (2) the maximum rate of rainfall occurs during the time of concentration. The time of concentration, t/c, is defined as the flow time from the most remote point in the drainage area to the point in question.

It should be pointed out here that this method relies on average rainfall intensities prevailing over the time of concentration and that these average intensities have no relation to the actual rainfall pattern during the storm.

It should be also noted that the "rational method" is generally applicable to urban areas of less that 13 square kilometres and for larger areas, the application of the hydrograph method is usually warranted.

Expected Flow - "Q"

It is noted here that the assignment of the units c.c./sec. (cubic cm per second) is satisfactory to this equation since 7.0 c.c. /sec. equals one cm of rainfall in one hour over an area of one square meter.

Run-off Co-efficient - "R"

The coefficient can be further defined as the ratio between the maximum rate of run-off from the area and the average rate of rainfall on the area during the time of concentration. This coefficient cannot be exactly determined since it includes the influence of a number of variables, such as infiltration capacity of the soil, depression storage and interception of vegetation. The closer the area comes to being impervious, the more reasonable the selection of "C" becomes, since for highly impervious area "R" approaches unity. For these reasons mentioned, the run-off coefficient "R" is the component of the rational formula which requires the greatest exercise of judgement by the design engineer.

A partial list of coefficients presently in use are:

Character of Surface	Run-off Coefficients
Asphalt	0.70 - 0.95
Drives & Walks	0.75 - 0.85
Lawns - Sandy Soil	0.10 - 0.20
Lawns - Heavy Soil	0.13 - 0.30

Often municipalities will use an "average" coefficient where more than one type of surface is present in a particular drainage area. Some of those in use are:

Description of Area	Run-off Coefficients
Single Family	0.30 - 0.50
Semi-detached	0.40 - 0.50
Apartments	0.50 - 0.75
Industry	0.50 - 0.90
Parks, green belt	0.10 - 0.25

Rainfall Intensity - "i"

Rainfall behaviour varies with location and topography. The coefficient "i" is commonly determined from an equation such as:

$$i = \frac{x}{t + y}$$

where i = rainfall intensity mm/hr
t = time of concentration in minutes
x,y = constants determined from previous records

The relationship $i = \frac{x}{t + y}$ is hyperbolic

In practice, there is usually an upper limit on "i" which corresponds to a value to t_c from 10-15 minutes in urban areas and as low as 5 minutes for industrial or highly built-up areas.

For urban storm sewers, the time of concentration consists of inlet time, or the time required for run-off to flow over the surface from the most remote point in the drainage area to the nearest inlet plus the time of flow in the sewer from the uppermost point in the sewer to the point in consideration. The time of flow in the sewers is usually the average full-flow velocity expected for the particular sewer size, shape and roughness factor (i.e. hydraulics of the sewer).

Stormwater Management

In designated stormwater storage areas, the flow rate entering the storm sewers can be restricted or controlled by installing an inlet control device (ICD's) or by reducing the catchbasin lead size. The invert and size/shape of the ICD is critical with respect to the controlled release rate entering the sewer.

Stormwater Quality

In order to maintain suitable water quality, i.e., control sediments/contaminants from entering the storm sewers and ultimately the receiving body of water, some devices commonly used include:

i) Temporary Filter Cloth: A filter cloth is placed between the catchbasin grate and catchbasin frame. This is commonly done during the construction period to control sedimentation runoff.

ii) Goss Trapp: This prefabricated device is sometimes installed in catchbasins to minimize the amount of oil and other suspended particles from entering the storm sewers.

Protection Against Basement Flooding

Inlet Control Devices (ICD's) are also commonly installed in catchbasins to control the amount of stormwater flows entering the storm sewers and consequently to avoid surcharging or back up of storm flows from sewers into basement via. house connections.

Catch Basins

One of the main purposes of a catch basin is to drain away surplus water from the road allowance. To accomplish this, a pair of catch basins on either side of a street area installed, spaced at intervals of 75 to 90 metres.

Until recently, it was customary that the designer would merely show that standard catch basins were to be installed at the points indicated. Today we have several different types of catch basins. These variations were all brought about by observing problems in the field and the designer must consider the conditions likely to prevail and choose the type of catch basin that will perform best under a given situation.

Catch basins with throttling devices can be purchases for use in areas designated as storm storage areas.

Construction

A very important part of the work is the construction of the sewers.

To operate efficiently and avoid unnecessary maintenance and general operating problems, sewers must be laid to the design line and grade and remain o line and grade. This is usually accomplished with grade stakes to which batter and string lines are set.

Particularly on larger pipes some contractors will use a transit to keep the pipe on line. Today a Laser Beam is available which is set in the downstream manhole and projects through the pipe to a target placed in the bell of the pipe being set. This provides for very rapid progress of the work and contractors who have used this method claim to have achieved at least twice the daily production.

Bedding of the pipe is of equal importance to insure the pipe does not settle after backfilling. This includes the compaction of suitable granular material around and at least 300mm above the pipe to insure that first, the walls of the pipe are supported to the trench sides so that its structure can carry the load to be placed on it, that it will not be displaced out of line and last, that it will not be damaged during the backfilling operations. (When completed, a good job should look like a gun barrel.)

Points an inspector should be watching for are as follows:

1. The bedding must be to grade and a hole dug out for the bell to prevent bridging of the pipe from bell to bell on the bedding.

2. Making final adjustments to the pipe grade by placing pieces of wood under the bell or front edge of spigot piece but not removing when grade is set.

3. Poor or no compaction under, around and over the pipe.

4. Large boulders in the backfill even with some cover on the pipe.

5. Placing of concrete under inlet/outlet pipes between base of manhole and natural excavated ground.

6. Proper connection of house services at the main sewer and a proper plug in the end of the service, especially where connecting to an older sewer.

7. A very important duty of the inspector is to ensure that pipe joints are made properly.

8. Smaller pipes are usually put home by hand with a large bar. They must be fully driven home and not permitted to spring back. Larger pipes should be put home with a come along operated from inside the pipe. The bucket of a backhoe should not be used to home a pipe.

APPENDIX - LIST OF SYMBOLS AND ABBREVIATIONS

Q	=	discharge in m^3/s, MLD or L/s
V	=	velocity in metres per second
m/s	=	metres per second
A	=	cross sectional area in m^2
H	=	head loss in metres/kilometre
c	=	coefficient of friction
S	=	slope in metres/metre or metres/kilometre
m^3/s	=	cubic metres per second
MLD	=	million litres per day
L/s	=	litres per second
m	=	metres
m^2	=	square metres
sec.	=	second
θ	=	diameter in millimetres (mm)

SUBJECT:

CONSTRUCTION OF SEWERS
WATERMAINS AND ASSOCIATED
FACILITIES

TOPIC: 9

WATERMAIN DESIGN

OBJECTIVES:

The trainee will be able to:

1. Understand the fundamentals in the design of a Water Distribution System.

2. Understand the Construction Procedures including materials used for a watermain and its appurtenances.

WATERMAIN DESIGN

GENERAL

A watermain system is designed to adequately satisfy the water requirements for some combination of domestic, commercial and industrial users as well as fire-fighting purposes. The system should be capable of satisfying the maximum demand placed upon it.

A water system is composed of four main parts:

a) Source plus treatment - lake, river or well plus necessary purification and pumping facility.
b) Transmission line (feeder main) from treatment facility to distribution system.
c) Storage - in-ground reservoirs, elevated tanks or stand-pipes.
d) Distribution system to customers.

WATERMAIN DESIGN

The design of watermain for an intake to a treatment facility, a transmission line or for a water distribution system involves certain overall considerations such as flow requirements, hydraulic adequacy, structural adequacy and economic efficiency.

Flow Requirements

The required capacity of an intake is established from domestic demand, fire flow requirements, industrial and commercial needs all being for a given time period. This required capacity may also be used to size water treatment facility if such is deemed necessary.

In most cases, the transmission line from the treatment facility to the distribution system will be part of the major piping in the distribution system.

The capacity of the distribution system is determined on the basis of the local water needs of the municipality. Water needs are comprised of domestic, commercial and industrial needs and fire-fighting demand.

a) **Domestic Requirements**

Domestic consumption is evaluated from past records and metered water consumption. From the metered consumption the average day rate expressed in litres per capita per day (L/c/d) can be obtained by dividing the annual consumption in litres by the population and by the number of days in a year. These values vary considerably for different localities. Therefore, careful study must be made of past records, the type and pattern of water use, the physical and climatic condition of the area, expected trends in development and projected population values.

b) **Commercial and Industrial Flow Requirements**

It is necessary that individual studies of the water requirement for a specific industry be made because quantities of water used by industry and commercial establishments vary widely. It is most important that the magnitude and time of occurrence of the peak rate be known when considering commercial and industrial requirements, especially when the watermain is to be designed to serve residential consumers along with commercial and industrial consumers.

The time of occurrence of peak rate is of considerable importance since system peak hours generally coincide with residential peak period.

c) **Fire Demand**

The coincident draft of water during fire-fighting is usually considered to be equal to the maximum daily demand since the probability of maximum rate of water usage for community purposes occurring simultaneously with major conflagration is slight.

Annual volumes required for fire purposes are small but during periods of demand may be exceedingly large and may govern the design of the watermain.

For values of flow requirement for fire demand the requirements of the Insurers Advisory Organization could be followed. This Organization has detailed guidelines for determining the required fire flow.

An estimate of the fire flow required for a given fire area may be determined by the formula.

$$F = 15C\sqrt{A}$$

where F = the required fire flow in Imperial gallons per minute
C = coefficient related to the type of construction.

C = 1.5 for wood frame construction (structure essentially all combustible).

C = 1.0 for ordinary construction (brick or masonry walls, combustible interior).

C = 0.8 for non-combustible construction (unprotected metal structural components).

C = 0.6 for fire-resistive construction (fully protected frame, floors, roof).

NOTE: For types of construction that do not fall within the fire categories given, use a coefficient reflecting the differences. Such coefficients shall not be greater than 1.5 nor less than 0.6 and may be determined by interpolation.

A = the total floor area (including all stories, but excluding basements) in the building being considered. For fire-resistive buildings consider the 6 largest successive floor areas if the vertical openings are unprotected; if the vertical openings are properly protected, consider only the 3 largest successive floor areas.

The values obtained from the preceding formula may be adjusted as a result of other factors such as proximity of buildings, presence of complete automatic sprinkler protection etc., for these the designer should consult the Insurer's Advisory Organization.

d) **Design Flows - Conclusion**

The four design flows used to design watermains are, therefore:

1. peak residential flow rate
2. peak industrial flow rate
3. minimum hourly rate in combination with reservoir filling
4. maximum daily flow rate plus fire flow.

Local watermains are usually designed to meet maximum daily plus fire flow requirements or peak rate demand whichever is greater. Thus, for design purposes, the maximum daily flow for commercial, residential and industrial demand whichever is greater. Thus, for design purposes, the maximum daily flow for commercial, residential and industrial demand must be added to the required fire flow to determine the required capacity of the main if the peak rate is less, which is the usual case.

The factors used to determine the maximum day consumption and the peak rate consumption may vary with the size of the community, as noted in Table 9-1 following:

Table 9-1
Maximum Day and Rate Calculations

Population Range	Maximum Day Factor	Maximum Rate Factor
0 - 500	3.00	4.50
501 - 1,000	2.75	4.13
1,001 - 2,000	2.50	3.75
2,001 - 3,000	2.25	3.38
3,001 - 10,000	2.00	3.00
10,001 - 25,000	1.90	2.85
25,001 - 50,000	1.80	2.70
50,001 - 75,000	1.75	2.62
75,001 - 150,000	1.65	2.48
greater than 150,000	1.50	2.25

Example: Population - 1,500 persons
Average per capita demand - 500 L/c/d
Average demand - 1,500 x 500 = 750,000 L/d
Maximum day - 750,000 x 2.50 = 1,875,000 L/d
Maximum rate - 750,000 x 3.75 = 2,812,500 L/d

Hydraulic Design

a) Pressure Requirements

In designing watermains for a distribution system, pressure requirements for ordinary use and for fire-fighting must be considered. Capacity of the system to serve is a function of available pressure. The water must rise to the upper stories of buildings of normal height and must flow from hydrants.

The pressure or head at any location in the distribution system is equal to the sum of the static head exerted on the pipe walls, plus the dynamic head caused by the flowing water. The total pressure or head may be determined as follows:

$$H = \frac{100P}{\delta} + \frac{V^2}{2g}$$

where H = total head (metres of water)
$100P/\delta$ = pressure head (metres of water)
$V^2/2g$ = velocity head (metres of water)
P = gauge pressure (kPa)
δ = specific weight of water (1000 kg/m^3)
V = flow velocity (m/sec)
g = acceleration due to gravity (9.81 m/sec^2)

The energy grade line can easily be determined by adding the elevation of the watermain to the equation for the total head. Thus:

$$EGL = \frac{100P}{\delta} + \frac{V^2}{2g} + Z$$

where EGL = Elevation of Energy Grade Line (metres)
Z = Elevation of watermain

The hydraulic grade line can be calculated using the following equation:

$$HGL = \frac{100P}{\delta} + z$$

The concept of the hydraulic grade line is very important in the sizing of new watermains. An acceptable standard requires that minimum system pressure head shall not be less than 14 metres of water under a fire flow condition or less than 28 metres of water under normal operating conditions. Most feedermains are usually designed for pressures between 28 metres and 70 metres of water. These limits are set because a pressure higher than 70 metres can result in excessive leakage or watermain breaks. A pressure lower than 14 metres during fire flow can cause problems in the system as the fire truck pumps can pump the system dry. The very low or negative pressure so created may result in problems such as contamination, damage to water heaters, etc.

Large service areas or areas that have a large variation in topography may require that pressure zones be established to deliver water at the proper pressure to the consumers. Based on accepted practice, if the topography varies by more than 42 metres minus an allowance for head loss in the piping system, consideration should be given to the establishment of a new pressure zone. If a new pressure zone is not created, then the water pressures in the low lying areas would be excessive due to the high pressure required at the source to deliver water to the higher points of the service areas.

b) Flow and Sizing Watermain Pipes

When the flow rates have been estimated and the desired hydraulic grade line determined, the size of the watermain can be calculated.

As water flows in a pipe, friction forces are developed causing a loss in water pressure.

The capacity of a pipe flowing full can be determined by the use of the Hazen-Williams formula. This formula is presented as follows:

$V = 0.355\ C\ D^{0.63}\ S^{0.54}$ where V = mean velocity (m/sec)
C = roughness coefficient
D = pipe diameter (m)
S = hydraulic gradient (m/m)

The value of the roughness coefficient C, is often assumed to be 120 for most piping materials. Manufacturer's claims of C values must be judged by the pipe designer.

The following C values should be used regardless of material in hydraulic sizing of a water distribution system:

150mm diameter - C = 100
200 - 250mm diameter - C = 110
300 - 600mm diameter - C = 120
Over 600mm diameter - C = 130

The Hazen-Williams formula can be expressed in terms of the flow in a watermain as follows:

$Q = 0.355\ C\ D^{0.63}\ S^{0.54}\ A$
where Q = flow (m^3/sec)
A = area of pipe (m^2)

The following example illustrates the use of these equations in the sizing of watermains.

A town wishes to supply water to proposed development area. The design flow has been previously determined to be 5,000,000 L/d. The connection to the existing water distribution system will be made at a location where the static pressure is 400 kPa and the watermain elevation is 150 metres. The length of the proposed watermain is 1,000 metres, the ground elevation at the end of the proposed watermain is 160 metres. The required head at the end of the watermain is 28 metres of water. What size of watermain is required?

Required Hydraulic Gradeline = 160 + 28 = 188 metres
Hydraulic Grade Line at connection point

$$H = \frac{100(400\ kPa)}{1000\ kg/m} + 150 = 190 \text{ metres}$$

Allowable Head Loss = 190 - 188 = 2 metres

$$\text{Allowable Hydraulic Gradient} = \frac{2}{1000} = 0.002 \text{ m/m}$$

Required flow = 5,000,000 L/d
= 5,000 m^3/day
= 0.05787 m^3/sec

Re-arranging Hazen-Williams and assuming C = 120

$$D^{0.63}A = \frac{Q}{0.355 \times C \times S^{0.54}}$$

$$= \frac{0.05787}{0.355 \times 120 \times (.002)^{0.54}}$$

$$= 0.0389$$

Try various pipe sizes:

Size	D(m)	$D^{0.63}$	$A(m^2)$	$D^{0.63}A$	Remarks
150 mm	0.15	0.3026	0.01767	0.00535	Too small
300 mm	0.30	0.4684	0.07068	0.0331	Too small
400 mm	0.40	0.5614	0.15904	0.0893	Too large
350 mm	0.35	0.5161	0.09621	0.0497	O.K.

Check for Flow Velocity

$$V = \frac{Q}{A}$$

$$= \frac{0.05789}{0.09621} = .602 \text{ m/sec}$$

A 350mm watermain would suffice
A check must now be made to determine if this size of watermain is commonly used or readily available.

c) Losses Due to Fittings

Pressure losses due to turbulent flow in fittings should also be considered in the design of watermains. In some cases, these losses may be substantial. Changes in direction of flow, flow through valves, etc. all increase the pressure loss in a watermain.

The pressure loss due to fittings is usually expressed in terms of the velocity head of the flow. There are many reference books available which give head loss coefficients for various types of fittings. It is recommended that the watermain designer refer to a good hydraulic reference book to obtain the appropriate values of head loss coefficients for valves and fittings.

Selection of Pipe Materials

There is a wide range of pipe materials currently available for the construction of watermains. Some of these materials apply only for a specific application, others are satisfactory for more general use.

The most common materials in use are:

a) Asbestos Cement
b) Ductile Iron
c) Polyethylene (P.E.)
d) Polyvinyl Chloride (P.V.C.)
e) Reinforced Concrete
f) Steel

Factors to be considered in the selection of the pipe material are:

a) Cost
b) Availability in size and pressure class
c) Ease of laying and jointing
d) Structural capacity in place
e) Facility for thrust restraint
f) Corrosion resistance
g) Bedding requirements
h) Type of Joints
i) Electrical conductivity
j) Hydraulic Capacity

Each one of these factors should be investigated in detail prior to the selection of the piping material.

a) Hydraulic Capacity

The hydraulic capacity of each of the watermain pipe material alternatives should be evaluated when selecting the pipe material. There are a number of claims by various manufacturers regarding the Hazen-Williams roughness coefficient C, for each pipe material. The designer must rely upon the information that he feels is most applicable.

From an examination of the Hazen-Williams formula, it can be seen that the conveyance factor (K) of a watermain for any flow condition can be defined as follows:

$K = CD^{2.63}$ ($\pi/4$ may be ignored in these comparisons as it is a common multiplier)

Using this equation, various pipe materials can be readily compared with regard to hydraulic capacity. The following table indicates the hydraulic capacity of alternative 300mm diameter pipe materials which must be capable of withstanding an internal pressure of 1035 kPa.

Hydraulic Comparison of 300mm Diameter Pipes

($K = CD^{2.63}$, D in metres)

Nominal Size (mm)	Actual I.D. (mm)	Hazen-Williams C	Conveyance Factor K
300mm AC Class 150 AWWA C400	304.8	120	5.27
300mm D.I. Class 50 ANSI A21.51 ANSI A21.4	316.4	120	5.82
315mm P.E. Series 160 CGSB 41-GP-25	250.4	120	3.14
300mm P.V.C. Class 150 AWWA C-900	298.0	120	4.97
300mm R. Conc. Class 150 AWWA C-303	304.8	120	5.27
300mm Steel AWWA C-202 AWWA C-205	296.8	120	4.92

The larger the value of the conveyance factor K, the greater the hydraulic capacity of the pipe.

b) Pipe Materials and Pressure Class Availability

The various pipe materials identified previously are available in a wide range of pressure classes. In order to simplify the selection of the proper pipe material and pressure class, the following pages list the types of pipe available for use in a water distribution system.

This list has been prepared with imperial designations as the pipe is still being manufactured to imperial units.

Pipe Materials and Pressure Class Availability

Pipe Type	Pressure Class (PSI)	Sizes Available (in.)
Asbestos Cement	Class 100,150,200	6,8,10,12,14,16,18,20,24
Asbestos Cement Transmission Pipe	Class 300,350	8,10,12,14,15,16,18,20,21,24,27,30,33,36
Ductile Iron	Class 50,51,52,53,54	4,6,8,10,12,14,16,18,20,24
Polyethylene	Class 60,80,100,125,145,160,200	3,4
Polyethylene	Class 45,60,80,100,125,145,160,200	6,8
Polyethylene	Class 36,45,60,80,100,125,160	10,11,12
Polyethylene	Class 36,45,60,80,100,125,160	14,16
Polyethylene	Class 36,45,60,80,100,125,145	18,20
Polyethylene	Class 36,45,60,80,100	22,24,28
Polyethylene	Class 36,45,60,80	32,36
Polyethylene	Class 36,45,60	40,48
P.V.C. AWWA C-900	Class 100,150	4,6,8,10,12
P.V.C. Series Pipe	Class 125,160	4,6,8,10,12

Pipe Type	Pressure Class PSI	Sizes Available (in.)
P.V.C. with Fibreglass Coatings	Class 150,200	4,6,8,10,12
Reinforced Concrete AWWA C-301(L)	Class 100,120,140,160,180, 200,220,240	16,18,20
Reinforced Concrete AWWA C-301(L)	Class 100,120,140,160,180 200,220	24,30
Reinforced Concrete AWWA C-301(L)	Class 80,100,120,140,160, 180,200	36
Reinforced Concrete AWWA C-301(L)	Class 60,80,100,120,140, 160,180,200	42,48
Reinforced Concrete AWWA C-301(E)	Class 60,80,100,120,140, 160,180,200,220	48
Reinforced Concrete AWWA C-301(E)	Class 60,80,100,120,140, 160,180,200,220,240	54,60,66
Reinforced Concrete AWWA C-301(E)	Class 40,60,80,100,120, 140,160,180,200,220	72
Reinforced Concrete AWWA C-303	Class 175,200,225,250	16,18
Reinforced Concrete AWWA C-303	Class 130,150,175,200,225,250	20
Reinforced Concrete AWWA C-303	Class 100,125,150,175,200, 225,250,275	24,30

c) Structural Adequacy

All watermains are structures. The pressure classification of a pipe is important for the time that the pipe is filled with water under pressure, but consideration must also be given to the structural adequacy of the pipe to resist the external loads upon it when out of service or when under construction.

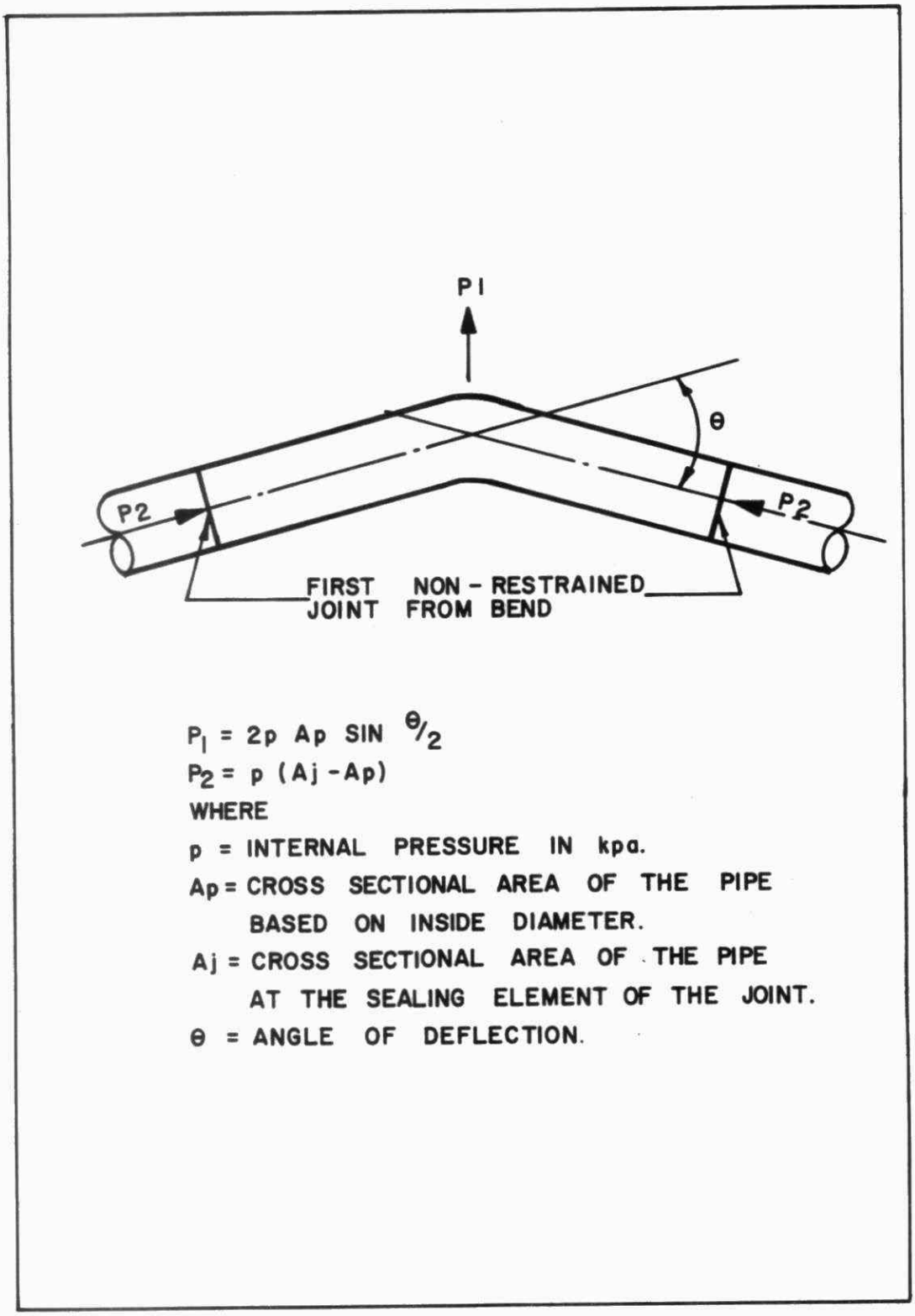
Thought must, therefore, be given to the design of the pipe in the trench, whether the pipe is rigid or flexible and whether the trench is an embarkment or a trench condition. Bedding, backfill, compaction and depth of cover are other factors to be considered.

d) Restraining Methods

A watermain is a pressure conduit which transmits a force along its length. In a straight line condition, this force is balanced by an equal force in the opposite direction. unbalanced conditions occur at bends, tees, valves, etc. and methods of restraining these unbalanced forces are required at these locations. It is recommended that a structural engineer design the thrust block, restrained joints or valve chamber used for restraint.

The active force diagram and their most common methods of restraint are detailed on Figures 9-1 and 9-2 following.

Figure 9-1 Diagram of Active Forces



$P_1 = 2p\ Ap\ \sin\ \theta/2$

$P_2 = p\ (Aj - Ap)$

WHERE

p = INTERNAL PRESSURE IN kpa.

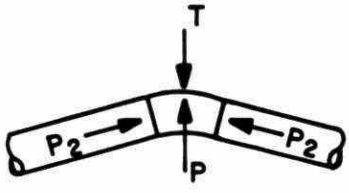
Ap = CROSS SECTIONAL AREA OF THE PIPE BASED ON INSIDE DIAMETER.

Aj = CROSS SECTIONAL AREA OF THE PIPE AT THE SEALING ELEMENT OF THE JOINT.

θ = ANGLE OF DEFLECTION.

Figure 9-2 Most Commonly Used Restraining Methods

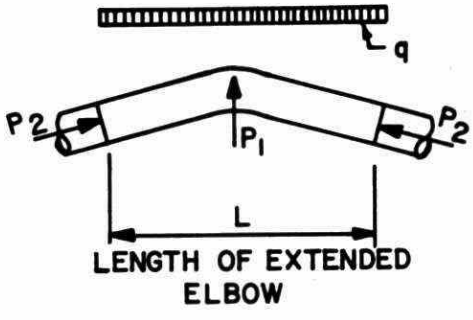
A. THRUST BLOCK



ALL ACTIVE FORCES ARE RESISTED AT THE BEND BY PASSIVE EARTH PRESSURE.

$T = 2p\ Aj\ SIN\ \theta/2$

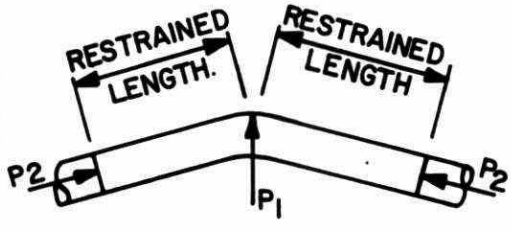
B. EXTENDED ELBOW
(FOR SMALL BENDS)



ALL ACTIVE FORCES ARE RESISTED BY FRICTION BETWEEN THE SOIL AND THE EXTENDED ELBOW IN THE DIRECTION OF THE BISECTOR.
EXTENDED ELBOW MUST BE DESIGNED TO TAKE BENDING MOMENTS AND SHEAR.

$T = ql = 2p\ Aj\ SIN\ \theta/2$

C. RESTRAINED JOINT



ALL ACTIVE FORCES ARE RESISTED ALONG THE PIPE BY FRICTION BETWEEN THE SOIL AND THE PIPE.
PIPE MUST BE DESIGNED TO TAKE TENSION.
FORCE TO BE RESISTED ALONG EACH LEG IS

$F = p\ Aj$

SUBJECT:

CONSTRUCTION OF SEWERS WATERMAINS AND ASSOCIATED FACILITIES

TOPIC: 10

GEOTECHNICAL CONSIDERATIONS

OBJECTIVES:

The trainee will be able to:

1. Understand Glaciation.
2. Understand classification of soils.
3. Be aware of some of the properties of different soil types.

GEOTECHNICAL CONSIDERATIONS IN SEWER & WATERMAIN CONSTRUCTION

INTRODUCTION

For the engineer or technician engaged in the planning, design, construction and operation of civil works projects, the properties of soil, rock and groundwater, and their interaction with earthworks and underground structures is of primary importance. These properties include structure, density, strength, compressibility and permeability.

Unfortunately, unlike other construction materials such as steel, concrete, timber, etc. we cannot standardize soil because it is non-homogeneous, and wide variations in the subsurface conditions may occur between the test hole locations over a project site.

The design and construction of all structures deserves a subsurface investigation and related geotechnical recommendations. A comprehensive geotechnical investigation report is a good investment - it buys insurance, economy in design and construction, and it reduces claims.

The design engineers should obtain and read the geotechnical report thoroughly, and then incorporate the recommendations into the construction drawings and project specifications. The construction manager should ensure that all geotechnical requirements for the project are carried out, and seek geotechnical engineering assistance throughout the project.

The father of soil mechanics, Dr. Karl Terzaghi stated,

> "... on account of the fact that there is no glory attached to the foundations and that the sources of success or failure are hidden deep in the ground, building foundations have always been treated as stepchildren and their acts or revenge for lack of attention can be very embarrassing."

and he has gone on to emphasize

> "...in earthwork engineering, success depends primarily on a clear perception of the uncertainties involved in the fundamental assumptions and on intelligently planned and consciously executed observations during construction. If the observations show that the real ... conditions are very different from what they were believed to be, the design must be changed before it is too late. These are the essential functions of soil mechanics in engineering practice.

GLACIAL GEOLOGY

For quantitative design and construction purposes, a geological description is inadequate. However, a geological description of the project area assists in correlating experiences between several sites. In a general sense it indicates the pattern of subsurface strata to be expected. The geotechnical engineer may carry out a study of references on local geology before a major subsurface strata to be expected. The geotechnical engineer may carry out a study of references on local geology before a major subsurface exploration program is undertaken.

The earth's crust has been undergoing changes for million of years. Earthquakes, volcanic action, floods, variable erosion and deposition, wind action and climatic factors produced the soil mantle which was derived originally from the underlying rock.

Many millions of years ago when the earth cooled from its gaseous state, it was composed of molten material. Further cooling formed a crust at the surface. Molten material or magma still exists at great depth below the earth's crust.

Igneous rocks are formed from magma which has cooled and hardened slowly. Sedimentary rocks are formed from weathering and disintegration of igneous rocks. The further weathering of fine rock fragments into sand, silt and clay and the deposition of these soils into depressions of the earth's crust, followed by consolidation, cementation and pressure formed sedimentary rocks such as shale, sandstone and limestone.

Metamorphic rocks result from changes in heat or pressure which alter other igneous or sedimentary rocks. Heat leads to recrystallization of the original rocks and pressure results in reorientation of the crystals and produces a banded or folded effect. Typical examples of metaphorism are sandstone to quartzite, limestone to marble, and shale to slate or mica.

Glaciers

A glacier is a mass of ice which is formed by the accumulation and pressurized compaction of snow. Glaciers flow under the influence of gravity and/or extrusion.

In Ontario, we are especially interested in the effects of the glacial era as the entire province has been covered by at least two major ice sheets. The last glacial era is known as the Pleistocene Epoch. During this period, a known minimum of four advances and retreats of glaciers have occurred in Ontario. The most recent ice age is believed to have ended about 20,000 years ago. This most recent ice blanket reached into the south of Lakes Ontario and Erie. The Wisconsin glacier advanced from two main centres in Labrador and west of Hudson Bay. The glacier advanced as far south as Ohio. The direction of flow is determined by the landforms and striations left by the glacier. As the ice advanced, the drainage of the Great Lakes region was completely disrupted by the damming of the natural water flow to the east. The water level rose in the Great Lakes are until it found outlets to the south through the Mississippi and Mohawk River systems. After the recession of the glacier, the area around the Ottawa basin rebounded to its present elevation and water spilled down through the St. Lawrence River system.

Continental glaciers, because of their great size, tend to bury completely the area which they traversed. The weight of ice of 3000m thickness in the centre of the glacier exerts an immense pressure of about 30 MPa on the lowest layer of ice, causing glacial flow. Glaciers erode rock by abrasion or plucking. Generally, the ice mass gouges out the softer materials and leaves harder rock masses standing. Since the softer rocks are more easily gouged out by the ice, they tend to form basins which form lakes. The Great Lakes are an example of this.

As the thousands of winters and their load to the snow to the top of the ice, the ice is kept moving outward.

As the summers become longer again, the sun melted more ice than was replaced during the winters. The net effect was that the outer edges of the ice mass as well as the top of the glacier melted. The runoff from these melting periods would form rivers streaming from the ice mass. This water also helps for the land by erosion near the ice and by depositing solids further downstream.

Deposition

A ground moraine or till sheet is formed when materials are picked up by the flowing glacier, transported beneath the ice and redeposited as a compacted unsorted mixture of boulders, gravels and rock dust.

A drumlin is another basic landform moulded under the advancing ice and containing glacial till material in an oval ridge. As the temperature drops, the advance of the ice front is halted sufficiently to cause melting. Till material carried forward by the ice is dumped in a series of hills along the front of the ice called end moraines. If the ice advances and retreats several times, a succession of small ridges is formed, known as recessional moraines. When the ice melts without advancing again a fairly level ablation till is left. This consists generally of boulders, sand, silt and clay all intermixed.

The glacial melt water produces streams and rivers. Assorted materials deposited in flat plains beyond the ice perimeter are called outwash plains. When melt waters are contained within valley walls, the higher velocity flow carries coarse gravels deposited as spillways. Streams flowing in tunnels beneath the melting ice blocks may have removed silt and clay and left long ridges containing coarse sand and gravel materials called eskers. Melt waters rushing along the edge of the glacier dumped gravel, sand and silt against the ice front. When the ice finally melted and support was withdrawn, these mounds of containing alternate layers of silt and clay are known as kames. Lake bottom deposits containing alternate layers of silt and clay are known as varved clays. Where the glacial river is emptied into lakes, broad fan shaped deltas were formed. These sediments grade rapidly from coarse to fine, and consist of gravel, sand and silt.

When the geotechnical engineer can correlate topographical features to glacial formations, then potential subsoil problems may be anticipated and avoided.

SUBSURFACE INVESTIGATION

A subsurface investigation involves the appraisal of the general subsurface conditions at the project site by analysis of information gained by such methods as geological and geophysical surveys, insitu testing, boring and sampling, visual inspection, local experience, laboratory testing of samples of the subsurface materials and groundwater observations and measurements.

The subsurface investigation is the first and most important step in any underground structure or earthworks project. Such an investigation should be carried out for all structures, even modest ones, before design is undertaken or a building permit is issued.

It is important that subsurface investigations be carried out under the direction of geotechnical engineers and personnel with knowledge and experience in planning and executing such investigations. It is desirable that drilling crews be experienced specifically in borings for geotechnical explorations. The primary objective of a subsurface investigation is to determine as accurately as may be required:

a) the nature and sequence of soil strata,
b) the groundwater conditions at the site,
c) the physical properties of the soils and rock underlying the site,
d) the mechanical properties such as strength and compressibility of the different soil or rock strata, and
e) other specific information, when needed, such as geochemical composition of the soil and groundwater, and characteristics of foundations of adjacent structures.

Site investigations should be organized in such a way that all possible information be obtained that will provide a thorough understanding of the subsurface conditions and probable foundation behaviour.

Particular attention should be paid to our present approach in a subsurface investigation, whereby we inspect a very small soil sample versus the very large and varying soil mass on site, and subsequently attempt to predict the behaviour beneath the proposed structure.

This approach requires knowledge, experience, sound judgement and to some extent feeling and luck.

The field inspector, at the time of construction, has the benefit of seeing all the soil instead of only the sample obtained in a borehole investigation.

All variations from predicted soil conditions should be reported immediately by the field inspector of the geotechnical engineer.

Field Testing Methods

i) Standard Penetration Test:

This test is carried out by driving a standard split spoon sampler of fixed dimensions under a constant driving energy, usually a weight dropped a uniform number of times, over a uniform distance. A split spoon sampler is usually of small diameter and is primarily designed to extract a disturbed sample.

ii) Dynamic Cone Test:

Instead of a tube-type sampler, a rod with a standard conical end is driven into the soil with known energy as with the split spoon test. No sample is available.

iii) Static Cone Test:

This test differs from the dynamic cone test since the energy is applied as a constant pressure rather than a series of regulated impacts.

iv) Plate Bearing Test:

This is usually carried out in an open pit with a steel plate loaded in equal increments. Settlement is then measured after each weight is added, allowing appropriate intervals between increments.

v) Vane Test:

This is used in soft clays to determine in-place shear strength. The field vane usually consists of four blades at 90° to each other which can be attached to the end of a rod and then inserted in the borehole and carefully pushed into the soft soil. Then the torque required to rotate the shaft is measured, from which the shear strength can be derived.

vi) Pressure Meter Test:

This consists of a metal cylinder covered with an inflatable rubber membrane, which can be activated by gas or liquid pressure from a measured source. This allows the determination of the load and deformation characteristics of the soil or rock in the open boreholes.

End effects are minimized by the use of the guard cells at each end of the test cylinder, which are expanded tightly to the wall of the borehole prior to testing.

vii) Permeability Test:

This involves the adding of water to soil to determine the soil's ability to pass water through the soil mass. Such tests include falling and constant head tests in cased or uncased boreholes. This could include basic percolation tests in shallow test holes as well as sophisticated full scale pumping tests.

SOIL CLASSIFICATION

Soil is that portion of the earth's crust which is fragmentary, or such that some individual particles of a dried sample may be readily separated by agitation in water; it includes boulders, cobbles, gravel, sand, silt, clay and organic matter.

There are three major groups of soils:

a) Coarse Grained Soils - particles which are large enough to be visible to the makes eye. They include gravels and sands, which are larger than the No. 200 sieve (0.075 mm). These are generally referred to a cohesionless or non-cohesive soils.
b) Fine Grained Soils - silts and clays are cohesive.
c) Organic Soils - soils having a high natural organic content.

Soils are identified and classified according to their particle size and distribution for coarse-grained soils, and their plasticity for fine grained soils, which are based on the Unified Soil Classification System.

Visual Classification

Generally, there is little problem in recognizing coarse grained soils which, as explained above, comprise sands, gravel and cobbles. Some confusion invariably occurs distinguishing silts from clays. Silts are frequently classified incorrectly as clay. This error can lead to construction problems since the behaviour of silt, both in an excavation base and a construction backfill material, is entirely different from either clay or sand.

Based on a visual inspection of several samples taken from a site, it should be obvious which soils are the coarse grained sands and gravel. Even fairly fine sands are easily recognized. The difference between silt and clay is not as obvious.

In the field, it is usual to distinguish between silts and clays using a dilatancy test. A wet "pat" of soil is shaken in the palm of the hand and if the soil exhibits dilatancy, it will show free water on the surface. Moreover, this free water will disappear when the soil is squeezed and the soil will then become stiff and crumbly. Soils which exhibit dilatancy are normally silts. Clays will not exhibit dilatancy.

The sense of touch can be used to distinguish silts from clays and from sands, whereas sands have a gritty feel and the grains visible to the naked eye. Silts have a rough, gritty texture and clay has a smooth plastic feel. Clay, in fact, can be rolled into very thin threads without breaking. Clay sticks to the fingers and it dries slowly, but silt dries fairly quickly and can be dusted off leaving only a stain. It is not always possible to categorize soils as a strict sand, silt or clay type. Frequently, any material will consist of a combination of either type, although normally one is predominant. For example, a silty clay is a predominantly clay material with some silt content, ie. it will not exhibit dilatancy and will feel mainly smooth and plastic, however, with some evidence of a rough texture. On the other hand, a clayey silt will show dilatancy although it will also be plastic to some degree ie. it can be rolled in the fingers.

Laboratory Tests

In addition to visually identifying soils in the field, it is important to carry our laboratory tests such as sieve and hydrometer analyses for soil classification. Soil classification may be carried out on the basis of grain size diameter as follows:

Coarse Gravel	75 mm to 20 mm
Fine Gravel	6 mm to 2 mm
Coarse Sand	2 mm to 0.60 mm
Medium Sand	0.60 mm to 0.20 mm
Find Sand	0.20 mm to 0.60 mm
Silt	0.60 mm to 0.002 mm
Clay	less than 0.002 mm

The above divisions are not exact. The characteristics of a soil are indicated not only by grain size but also by grain shape such as angular or rounded, water content, and grading such as well or poorly graded.

It is important to be able to distinguish the differences between various soils such as clayey silt, sand and medium gravel. It should be noted that a grading curve will also provide an indication of the uniformity of the material. For example, a poorly graded or uniform sand being essentially one size eg. medium sand. The significance of the uniformity of soils as it affects compaction is discussed in a later section.

Imported Granular Fills

There are many types of imported soils used in construction. Generally, the materials are granular or cohesionless since they are easier to excavate from borrow pits, easier to handle and compact, etc. Depending on the function of the imported fill, material can vary from a Granular "A" to a sand cushion or Granular "C. By definition, a Granular "A" material is a superior type of granular soil having closer specified tolerances regarding particle size than either a Granular "B" or Granular "C". It should also not be overlooked that Granular "A", contains a specified small portion of crushed or angular stone. Granular "A", therefore should be used in the top course of a road since it has superior strength properties compared to the underlying sub-base Granular "B". Placed as pipe bedding, etc.

There is also confusion as to the meaning of 20 mm crushed stone. Where specified as 20 mm clear crushed stone, it contains simply angular crushed particles of stone at 20 mm size. Be definition, it is poorly graded. On the other hand, 20 mm crusher run stone contains angular crushed particles from 20 mm down to dust, ie. well graded.

In most of Ontario where there is a plentiful supply of granular soils they are also generally relatively clean that is, free from clay and silt contamination, and they are reasonably well graded and not just predominantly one size. For the most part, excavated sand and gravel from borrow pits are at a water content which enables the materials to be repacked fairly easily with minimum efforts.

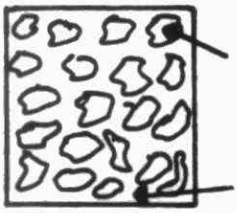
COMPACTION

Definition

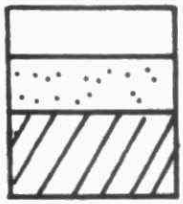
Soil compaction is the process of forcing soil particles closer together. This is accomplished by eliminating, or at least reducing, the air voids in a given volume of soil.

To illustrate:

A soil mass consist of:



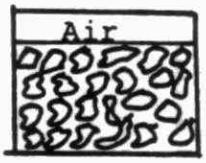
Pore Space



The moisture content is the most important factor for obviously, if the soil is saturated it cannot be compacted, since the voids are filled with water. Similarly, if the sample is day, the film of water around the soil particle is missing, or is too thin to lubricate the particles when the compactive force is applied.

Water and Air, large voids

Mass before force applied



Mass after force applied

Water and Air, but much smaller voids

The objective in compacting soil is to improve its properties, in particular to:

a) increase its strength and bearing capacity;
b) reduce its compressibility and thereby minimize settlement;
c) decrease its ability to absorb water.

Compaction Testing

The usual method of determining the compaction achievement in the field is to measure the in place dry density and to relate it to a laboratory dry density for the same material. The laboratory dry density is established by the Proctor Test.

i) Rubber Balloon Method:

A hole is dug into the fill to be tested. The soil removed from the hole is weighed and its moisture content determined. The volume of the hole is then measured by pumping a water filled rubber balloon tightly into the hole from a graduated cylinder and thus the density is determined.

ii) Sand Cone Method:

The only difference in this method from the balloon method is that the volume of the hole is determined by pouring sand of known unit weight into the hole.

iii) Nuclear Method:

Nuclear moisture density gauges, although available since the early 1950's, have only been used extensively since the 1970's.

They contain a small radioactive source which emits gamma rays to measure density, and neutrons to measure moisture.

In the field, the gauge is placed on the surface of the soil to be tested and one or more 1 minute counts are taken on both density and moisture channels. These counts are divided by the daily standard count obtained every morning by checking the gauge against a standard block. The resulting count ratio is referred to a calibration chart which indicates the respective wet density or moisture.

The field dry density is determined from the natural moisture content and the bulk unit weight. The dry density is related to the Proctor value as with conventional testing equipment to obtain the percentage compaction.

Advantages & Disadvantages of the Various Testing Methods

Sand Cone

Advantages	Disadvantages
a) very accurate if meticulously executed and sand properly calibrated	a) slow
b) gives technician "feeling" of soil and has visual examination built-in	b) subject to calculating errors
c) permits stone correction	c) moving and vibrating equipment has to be kept away
d) inexpensive	d) requires numerous pieces of equipment
	e) sensitive to weather influences
	f) requires separate moisture determination

Balloon Apparatus

Advantages	Disadvantages
a) faster than sand cone	a) requires skill and experience in pumping up balloon without rupturing it
b) gives technician "feeling" of soil and has visual examination built-in	b) in connection with a), there is danger that it will not be pumped up enough resulting in erroneous (high) densities
c) permits stone correction	c) should not be used in Granular "A", crusher run and similar materials
d) can be used while equipment working nearby	d) requires separate moisture determination
e) relatively inexpensive	

Nuclear Gauge

Advantages	Disadvantages
a) fast (more results obtained)	a) stone correction not possible
b) allows moisture as well as density determination	b) moisture channel not always reliable
c) can be used while equipment working nearby	c) does not give "feeling" of soil and does not allow visual examination without extra effort
d) weather resistant	d) requires knowledgeable operator
e) non-destructive testing (if necessary)	e) electronic components subject to breakdown
f) density "profile" can be taken with recent models	f) expensive

Note: All the above tests depend on an accurate identification of the soil being tested to ensure that the Proctor density used is representative of the soil being tested.

Factors Influencing Compaction

The main factors influencing the compaction of soil are moisture content (which is probably the most important), compactive effort, temperature, and grain size.

i) Moisture Content:

At a low moisture content, the soil is stiff and difficult to compress. When water is added, the individual particles and/or lumps become coated with a film of water which acts as a lubricant. The particles can slide along each other more easily. Up to a certain point, additional water replaces air and aids the compaction process. However, after a relatively high degree of saturation is reached, the water will occupy space which would normally be filled by soil particles and density decreases.

ii) Compactive Effort:

For most types of soil and compaction equipment, increasing effort will result in increased density.

It must be remembered that increased effort does not necessarily produce an equivalent increase in density as a maximum density for any soil is related to soil type, moisture content and the type of compaction equipment. Trial sections where the number of compactive passes versus field density is analyzed will generally provide as much information as Proctor and field density tests.

iii) Temperature

Although a minor factor in field compaction, it can become significant in our region in winter time.

iv) Grain Size

The grading characteristics can have a significant effect on the efficiency, as well as the effectiveness of compaction. If a material is poorly graded or uniformly graded (a material of predominantly one size), it is difficult to compact. For example driveways are sometimes constructed with a 300 mm layer of clear, crushed stone. It is virtually impossible to compact such a layer. On the other hand, it is relatively easy to compact a Granular "A" material because of well graded particle sizes.

Validity of Insitu Density Measurement

The validity of insitu field density tests should be fully understood.

The maximum dry density is usually established in the laboratory by taking a representative sample of the soil and compacting it with a standard hammer into a mould at varying moisture contents. The maximum density thus obtained is termed the Proctor dry density. It is the density to which all field densities are related.

There are two laboratory Proctor density values, Standard Proctor dry density and Modified Proctor day density. They are both obtained with similar produces except that the modified value is determined by applying a greater standard compactive effort to put the material into the mould. Thus the Modified Proctor density is slightly higher than the Standard Proctor value. When specifying compaction values, therefore, it is important to ensure that you are relating to standard or modified values. Standard values are used in most instances.

It should be noted that the Proctor dry density is by no means the maximum density that can be achieved in the field. It must be appreciated that under ideal conditions, densities well in excess of the maximum Proctor value can frequently be achieved in the field.

It is worth spending some time understanding what is inferred by 90% Standard Proctor density or 100% Standard Proctor density.

Most granular materials, when simply end-dumped from a truck and without any type of compaction, will achieve a density of approximately 83% to 85% of the standard (laboratory) Proctor dry density. When compacted under ideal conditions, on the other hand, it is possible to achieve field densities of about 112% to 115% of the standard (laboratory) Proctor dry density. Clays and silts, when end-dumped without compaction, are generally in the range of 78% to 82% Proctor density.

It is known that the relative compaction of soils in the field can vary from about a minimum of 80% Proctor density to 115%. Other Proctor densities therefore, are completely erroneous.

It should also be remembered that the density measurements in the field and Proctor tests in the laboratory are far from an exact science. Compacted soils generally fall into five categories, namely, very poor (compaction less than 85%); poor (compaction between 85% to 90%); intermediate (compaction between 90% and 95%); good (compaction between 95% and 100%), and very good (in excess of 100%). It also follows that rejection of compaction fill (specified to be compacted to at least 95%) may not be justified because of a few density values of say 94% or even 93%. Because of variables, it is generally only possible, in practice, to estimate the relative densities to any accuracy of +-2%. Thus calculated values of as low as 93% may still be reasonable, particularly if the majority of values are in excess of 95%. When dealing with large fills and ample results, we have suggested, for example, that no more than 3% of the random value fall below the specified with no single result less than 5%.

Currently various papers have been published which suggest that soils with natural moisture contents in excess of optimum, and insitu densities at less than 95% Proctor may be satisfactorily recompacted to 98% of insitu density with adequate resistance to future long term settlement.

Compaction Equipment

There are mainly two types of compaction equipment presently available namely static and vibrating equipment.

The former category static includes all rollers such as smooth wheel, pneumatic tired, sheepsfoot, pack-all etc. These static rollers are usually effective in cohesive soils such as clays and silts.

Vibratory compaction includes most of the above mentioned rollers with a vibrating mechanism attached to the drum, as well as the small vibratory plate tampers. These rollers and tampers should be used on granular soils such as silts or sands and gravel.

Rammers are a third type of compaction equipment. However, their use is and should be restricted to fill in confined areas where the other compactors cannot be employed.

Frequently Encountered Compaction Problems

i) Construction Problems:

There are a number of problems which the contractors repeatedly run into, including incorrect type of compactor not suitable for job site and/or material; lifts of excessive thickness; underlying weak soil and moisture content of backfill too high for achievement of required density.

The depth of loose layers to be compacted should normally not exceed 300 mm and frequently, with small plate tampers, etc. should be less then 150mm to achieve a reasonable degree of compaction.

On some occasions, it is impossible to achieve a high degree of compaction with imported granular fills since the underlying native soil is weak. This can be overcome by using an initial lift of clear stone which, in effect, firms-up the weak soil to attain placement of additional lifts of granular fill.

As previously mentioned, moisture content control is critical if effective, efficient compaction is to be achieved. It may, for example, be desirable to protect stockpiled material with tarps to ensure that the moisture content is not increased during wet weather conditions. Conversely, in dry, hot periods, the addition of water is required. Once the material is in place and has been compacted to the specified limit, an increase or decrease in moisture conditions will not substantially reduce the relative density.

ii) Technical Problems:

In place densities are related to laboratory Proctor tests. Obviously, if the material changes, then a new laboratory Proctor value has to be obtained. Changing material is very common in Ontario since the gravel pit sources having been deposited during the last glacial melt era are heterogeneous in composition. Sometimes, these changes are subtle enough to escape the attention of the inspector, thus leading in inaccurate results.

After one or two meters of fill have been placed in layers the density results obtained at the exposed grade with either the balloon, sand cone or nuclear gauge may not represent the compaction for the total depth. In this case, test pits have to be dug to ensure adequate coverage.

TRENCH EXCAVATION

In most instances, problems arise during underground pipe installation with either excavation procedures, bedding requirements, or compaction of backfill. Bedding is required for two reasons, ie., stabilize the natural soil in the trench base and to provide adequate pipe support.

Trench Excavation Problems

Trench excavation can be undertaken, without any serious problems in almost any type of soil, provided the water table is below the trench base. On the other hand, where excavation is attempted below a high water table, considerations are much different. Before attempting to excavate, the location of the water table and the soil type should be determined. We can define these two parameters during a borehole investigation and subsequent test pit excavation.

The water table can be located by leaving the test holes open for about one day. The soil type can be identified, by examining bulk samples from the test pits.

Having established both the water table and soil type, the method of excavation in most soils is relatively straight forward.

In predominantly clayey soils, the level of the water table to normal service pipe installation is not important. Since the soil is virtually impervious, even with a high water table, excavation will be relatively straight forward, since little or no water will enter the excavation area. Any water percolation into the trench is normally surface run-off and can be handled easily with normal construction drainage, that is, pumping from sumps and ditches. In soft clays, although groundwater control is generally not a serious factor, heaving of trench bases can occur with trenches in excess of 3 meters depth. This problem can be overcome by sloping back the trench sides to flatter angles.

In sand, it is normally accepted that excavation to the level of the water table will be straight forward. In fact, it should, in most instances, be possible to excavate approximately 300 mm below the water table using perimeter ditches and filtered sump pumps. If the sand is very fine, then the temporary excavation may be extended to approximately 600 mm below the water table. Under these circumstances, it is suggested that the installation be undertaken in as short a section as practical, ie. short trench length excavated, the pipe installed and then backfilled before the next section is excavated. Conversely, if the sand happens to be a uniform medium or medium/coarse sand, than a maximum depth of 300 mm below the water table is suggested. Excavation below the above suggested limits without a positive method of controlling the water table will lead to serious problems because of increased groundwater flows. These positive methods include the use of wellpoints or excavation within the confines of interlocking steel sheet piling.

Attempts to excavate trenches more than 300 mm below the groundwater table in a sandy soil is a problem. At many sites, the natural water table is 1 to 2 meters below ground surface. Thus excavation in this area to depths in excess of about 2 or 3 meters require either wellpoints or sheeting. It should be appreciated that wellpoints in fact, lower the water table and therefore, the wellpoint tips should extend to a depth of at least 2 meters below proposed trench bottom. Sheeting, on the other hand, does not lower the water table. However, correctly installed, it prevents collapse of the trench sides and boiling of the base, when adequately designed and installed. A specialist dewatering contractor should be used.

The performance of excavations in wet silts is extremely difficult to predict. Several deep excavations have been undertaken without having to resort to sheeting. Wellpoints are not normally effective in dewatering fine silts since they have very low permeabilities. On the other hand, similar attempts to excavate in fine silts stratified with coarse sand seams to coarse silts have met with serious problems.

Trench Stabilization

The need for stabilization to firm-up the base of a service trench prior to placing the pipe bedding is generally a function of the type of soil and, to some extent, the expertise of the contractor. In most areas, the soil in trench bases will tend to be easily disturbed, particularly in excavations slightly below the water table in sands without using well points, or in soft clays and wet silts. The disturbance to the soil in trench bases can also be increased with the continual passage of workmen, ie. the bae becomes liverish.

In most cases where a base instability problem is anticipated, or occurs, slightly over excavating the trench to provide for a thin layer of well graded, angular, granular material, firms-up the disturbed soil and prevents further deterioration. Frequently success has been achieved with clear stone, since it is heavy and its angular fragments bind and cement together to produce a firm base. A 50 mm clear stone is suggested for this purpose in most instances. A 150 mm layer is adequate. It should be noted that 50 mm clear stone must be placed to a depth that can be driven into the poor base. If too deep a layer of clear stone is placed, the large voids that remain can fill at a later date causing settlement. Granular A,B or C materials do not normally provide a good material to stabilize the trench base because of the large percentage of finer material which becomes unstable when wet.

Trench Backfilling

Trench backfill is compacted in order to improve its properties and, in particular, to increase its strength and supporting properties, as well as reduce its compressibility and thereby minimize settlement. For obvious reasons, therefore, compaction of backfill is essential in trenches beneath settlement sensitive travelled streets, parking areas, etc. It is not as critical where services traverse non-settlement sensitive open areas such as parks and fields where the backfill does not have to support repeated traffic loads and settlement is not important.

Most pipes will normally be surrounded with a granular bedding material. Trenches are usually then backfilled, up to grade, with approved native material previously excavated from the trench to reduce differential heaving. The lower portion of the trench is backfilled with approved Granular C and the upper 1.2 meters with native material.

The significance of non-compacted backfill in trenches should be appreciated. Numerous studies have been performed on the settlement values associated with compacted fills. It can be assumed that any fill compacted to a minimum value of 95 percent Standard Proctor density will not result in settlement of any appreciable magnitude. Fill compacted to a lesser degree, say 90 percent, may result in settlement, as follows:

$$\frac{H}{A}\ (95\% - \text{actual compaction } \%)$$

Where H = total depth of fill in question
A = 1 or 2 for various soil types

Thus, 2 meters of fill, estimated to be compacted to approximately 90%, will settle approximately 50 mm. End dumped fill (compacted to 85%) will settle 100 mm or more in some cases. This is an empirical relationship developed from experience and numerous case histories with soils in Ontario. It obviously produces an approximate value, although it is sufficient to provide a guide as to the effects of inadequate compaction of fills. It should be noted that most fills, in normal service pipe installation, are somewhat less than total trench depth after deducting the granular bedding materials which has been placed over the pipe and the upper 600 mm of granular road base. Frequently, the actual depth of native material in a trench is limited to about 1.2 meters. Thus, even with less than properly compacted fills, settlements will be fairly small. In most cases, some compactive effort is applied to the fill either during placement or by construction traffic, thus the resulting settlements are almost negligible. Such settlements may be tolerated in granular roads, but not on paved roads, so it is a good idea to delay repaving, particularly with top roads, but not on paved roads, so it is a good idea to delay repaving, particularly with top asphalt for as long as possible. At least one freezing and thawing season is preferable.

Native soil is generally placed back in the trench. This continuity with the adjacent native soil is required to prevent severe differential frost heaving problems if an alternate granular material is substituted.

In many instances therefore, it is unfortunately necessary to accept the lesser of two evils, ie. knowing that some slight settlement will occur in trench backfill with native backfill depending on the total depth, or no settlement but severe differential frost heaving movements if compacted granular fill is substituted. It has been found in the majority of cases that the settlement of poorly compacted native fills may be less than 50 mm whereas differential heaving can be at least 100 mm. In addition, once the settlement of the fill has occurred generally within six months, little additionally movement will take place. On the other hand, differential frost heaving will occur repeatedly each year.

It follows, therefore, that backfilling with native backfill is a logical alternative. However, some caution should be exercised when the fill exceed 2 or 3 meters depth, because of potential settlements. In this instance, special efforts should be undertaken with attempts to compact the native soil, such as scarifying and discing to dry the soil out to a moisture content suitable for compaction. Alternatively, the trench fill below the frost zone could be an imported Granular C which is readily compacted.

Typical results of distress in pavements due to inadequately compacted trench backfill are often evident on the approach to bridge abutments and culvert pipes, etc., where the fill settles. However, the adjacent structures are rigid since they are constructed on foundations designed not to settle.

Finally, one or two special cases should be mentioned, such as, construction of services in peat areas and bedrock. Most of the problems associated with pipe installation in rock are attributed to pipe bedding because of poor distribution. It is worth mentioning however, that rock and rock fill are notable poor insulators. Theoretical calculations, assuming an average number of freezing index days for a typical Canadian winter, indicate that frost can penetrate rock about 2 meters. This can be compared with 1 meter for clay and 0.5 metre for fibrous peat. Pipe trenches which cross both peat and bedrock require adequate transition zones to minimize differential settlement and cracking.

Construction in peak should, if possible, be avoided, that is, the line should be re-routed to avoid deep areas of peat. If however, this cannot be avoided, then whenever possible, the peat should be entirely excavate to a competent, non-compressible soil and backfilled up to invert level with imported fill. It is frequently practical to only excavate and remove peat to depths 4 to 5 metres beyond which complete excavation and backfilling becomes economically marginal. However, peat has been excavated to depths of 8 meters where localized deep pockets exist rather than run the risk of settlement, etc.

Other muskeg treatments where deep deposits are encountered involve partial excavation and partial displacement of the organic by surcharging the soil with fill and inducing failure beneath the proposed roadway, thus removing the organic from the construction. This procedure is somewhat unreliable, since it cannot be carefully controlled and some muskeg will undoubtedly remain, resulting in future settlement and maintenance of the road.

Surcharging with blasting is also possible but is not normally recommended although many articles suggest this procedure in wilderness conditions.

As a last resort, attempts can be made to float the pipe in peat. This approach needs very careful engineering consideration, in particular, the finished condition in the trench load is placed on the peat underlying the sides of the pipe. For example, if the trench were to be backfilled with imported fill, then the actual load of the peat would be much heavier (fill density of 1800 kg/m^3 compacted to peat density of 1020 kg/m^3). A pipe could be installed in peat, for example, by compensating the loss of weight due to the void created by the pipe with some slag bedding. The trench would then have to be backfilled with the peat. The slag would also be beneficial in tending to prevent the pipe floating. It is also important to install the pipe and backfill in as short sections as possible, probably in the order of 10 to 20 metre sections. Obviously, this scheme, although it has proved to be practical and successful requires a great deal of trial and error and very close supervision at the onset of the project.

One spectacular failure of a pipe occurred recently when the sewer was placed in compressible organic soil and was subsequently surcharged with 2.5 meters of fill. The pipe settled immediately and was displaced laterally, as well. During construction and relaying, one section of the pipe has been displaced to such an extent that it was never recovered.

Geosynthetics

There have been recent and rapid advancements in the supply and design and application of geosynthetics in civil engineering structures.

Geotextiles may be defined as filter fabrics designed as a permeable system providing separation, filtration, drainage and/or reinforcement for the given soil and groundwater conditions.

Suitable test methods should be carried out to confirm the suppliers data on physical, mechanical, hydraulic and chemical properties, as follows:

a) Physical - fibre type (woven and non-woven), length and width of roll, weight.

b) Mechanical - tensile, tearing and bursting strength, puncture resistance, elongation, abrasion resistance, soil/fabric friction.

c) Hydraulic - water permeability, soil retention

d) Chemical - resistance to light, ultra-violet, chemicals, micro-organisms, high and low temperatures.

Geosynthetics may also be deemed as impermeable liner systems, as for protection of methane and leachate migration from adjacent landfills.

GROUNDWATER

The construction of underground services, buildings with deep foundations, tunnels, etc. frequently requires excavation into water bearing materials. The presence of water within soil increases its resistance to maintaining an open excavation. In order to prevent caving or sloughing of the excavation slope, dewatering of the water bearing soil may be required to provide a dry and safe working area.

In cases where an excavation is undertaken near a water bearing stratum under excessive hydrostatic pressure, heaving, blow outs and quick or flowing soil conditions may result in loss of the excavation. Depressuring of the saturated stratum by dewatering may be employed.

Dewatering systems must be carefully reviewed by the designers and specialists contractors for the local soil and groundwater conditions and to the depth of the excavation. A properly designed, installed and operated dewatering or depressuring system will:

a) intercept or control seepage entering into an excavation
b) increase the stability of excavated slopes and prevent slope failure
c) prevent quick, heaving or caving conditions from waterbearing strata adjacent or beneath the bottom of an excavation by reducing excessive hydrostatic pressures
d) reduce lateral loads and earth pressures on temporary supports such as sheeting and bracing
e) improve excavation and handling characteristics of soil

When groundwater control methods are used, consideration should be given to the effect on local water wells. Also, improper handling of pump discharge may lead to complaints from local residents.

Well point deep well systems must be pumped for a considerable length of time prior to excavation and can lower water levels in a wide area. A running record of water levels in the excavation area should be kept. This can be achieved by installing well points at key locations and observing the water levels regularly (prior, during and after excavation). This information can forewarn of any deficiencies in the effectiveness of the groundwater control system and be helpful in the evaluation of local well interference complaints.

Groundwater Control Methods

Groundwater control may be effective by two main methods; those that depend on normal drainage processes, and those that keep the water from entering the excavation. A combination of the two may also be used in some cases.

a) Collection Ditches and Sumps

This is usually for small excavations at or near water bearing strata where the soil or rock is not likely to slough or erode. The seepage is collected in ditches and removed from the excavation by sump pumps. This system is unlikely to be satisfactory where inflowing water causes quick or unstable soil conditions.

b) Sheeting and Open Pumping

Excavation into water bearing soils may be achieved by driving steel piling to sufficient depth around the perimeter of the excavation, removing the earth and pumping out seepage as it enters the excavation.

Rapid lowering of the water level within the excavation as compared to the normal water level outside of the excavation, can result in a build up of considerable hydrostatic pressure on the sheeting and on the bottom of the excavation. Sheeting must be adequately designed to withstand high earth pressures, braced and be driven to seepage cut-off depths. These can be determined from soil borehole logs. Sheet piling may also be used in conjunction with slurry trench cut-offs, cement or chemical grouts and cofferdams of various construction.

c) Well Points

This system usually includes an array of 50 mm diameter screened well points which are jetted in around the excavation at closely spaced centres of about 1.5m. The well points are attached to a common header pipe which is connected to a suction pump. A well point system is used effectively for temporary groundwater control in materials such as gravelly sands, silty sands and similar soils.

If the local groundwater level must be lowered more than 5m (the maximum effective lift of suction pumps) below ground level, two or more stages may be needed.

Where deep excavation into relatively low permeability soils such as silty sand producing small amounts of water are necessary, well points of a slightly larger diameter may be equipped with an aspirator or educator pump, with the pump intake near the bottom.

Vacuum and electro-osmosis methods may be used for dewatering silts and sandy silts where wells are ineffective in lowering the groundwater levels. The vacuum system is a well point system which is connected to a vacuum pump. By reducing the pressure in the water bearing strata water squeezes into the well point and is removed. Soils that are stabilized by the vacuum method remain completely saturated with water and severe shocks can produce a collapse of loose soil structures resulting in quick and flowing soil conditions.

Electro-osmosis simply utilizes will points as negative electrodes to attract water. Rods are driven into the soil to act as anodes. The well points are connected to a header pipe and to a suction pump. The water flow is induced into the well points by an electrical current applied to the rods. The total flow from a conventional well point system is to be significantly increased when electro-osmosis is used.

d) Deep Wells

These are usually 150 to 450 mm diameter screened wells surrounded by a sand and gravel filter located at a relatively wide spacing around the perimeter of the excavation. Water is pumped using a submersible or turbine pump. Deep wells are used effectively where high yield, highly permeable soil strata are encountered and where the depth of excavation below the local groundwater level is about 10m. Deep wells or bleeder wells may be the only practical means of controlling uplift pressures where water bearing strata underlie impervious layers.

e) Compressed Air

This method of groundwater control is used exclusively in tunnelling. The tunnel is sealed off and compressed air is injected at a rate of about 1 kPa per metre of water level above the base of the excavation. This method is usually used in conjunction with tunnel support structures.

FOUNDATIONS

Shallow Foundations

A shallow foundation generally derives its support from the soil or rock close to the lowest part of the building which it supports. The depth of the bearing area below the adjacent ground is usually about equal to or less than the width of the bearing area, and vertical loads on the sides of the foundation due to adhesion or friction may normally be neglected.

Shallow foundations include such common footing types as slabs, rafts, spread footings, strip footings, pads, mats and sills.

The design of a shallow foundation system normally requires that both bearing capacity and settlement be checked. Structural distress from settlement as evidenced by such occurrences as cracking, and distortion of doors and window frames, is a common experience. The drastic effects of a bearing capacity failure are rare except perhaps during construction where shallow temporary footings are frequently used with falsework.

Deep Foundations

A deep foundation is a foundation unit that provides support for a building by transferring loads either by end-bearing to a soil or rock at considerable depth below the building, or by adhesion or friction, or both, in the soil or rock in which it is placed.

Piles are the most common type of deep foundation.

Piles can be pre-manufactured or cast-in-place; they can be driven, jacked, jetted, screwed, bored, or excavated. They can be of wood, concrete or steel. Bored piles of large diameter are frequently referred to as caissons.

The quality of a deep foundation system is highly dependent on construction technique, on equipment and on workmanship. Such parameters cannot be qualified nor taken into account in normal design procedures. Consequently, it is high desirable to design deep foundations on the basis of in-situ load tests on actual foundation units.

Instead of the use of piles, there are a number of techniques which can be used to improve the strength and compressibility of subsoils, such as, vibrio processes, preloading, chemical injection, freezing, and electro-osmosis.

APPENDIX "A"

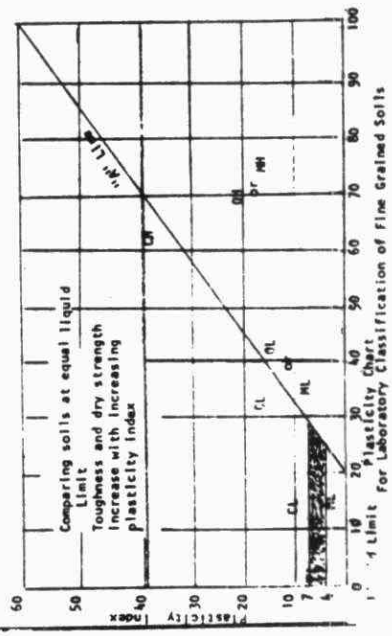
TABLE OF TYPICAL INSITU TESTS

Type of Test	Best Suited To	Type of Soil Not Applicable To	Properties that can be determined	Remarks
1-Standard Penetration test (SPT)	Sand	Clay	Qualitative evaluation of compactness. Qualitative comparison of subsoil stratification.	
2-Dynamic Cone Test	Sand and Gravel	Clay	Qualitative evaluation of compactness. Qualitative comparison of subsoil stratification.	
3-Static Cone Test	Sand		Continuous evaluation of density and strength of sands and gravel. Continuous evaluation of undrained shear strength in clays.	Test is best suited for the design of piles in sand. Tests in clay are reliable only when used in conjunction with vane tests.
4-Plate Bearing Test	Sand		Modulus of subgrade reaction. Ultimate bearing capacity.	Strictly applicable only if the deposit is uniform. Size effects must be considered in other cases.
5-Vane Test	Clay	Silt, Sand, Gravel	Undrained shear strength cu.	Test should be used with care particularly in fissured, varved and highly plastic clays.
6-Pressure meter Test	Soft Rock Sand		Ultimate bearing capacity and compressibility.	
7-Permeability Test	Sand and Gravel	Clay	Evaluation of co-efficient of permeability.	Variable head tests in boreholes have limited accuracy. Results reliable to one order of magnitude are obtained from long term, large scale pumping tests.

Unified Soil Classification
(including identification and description)

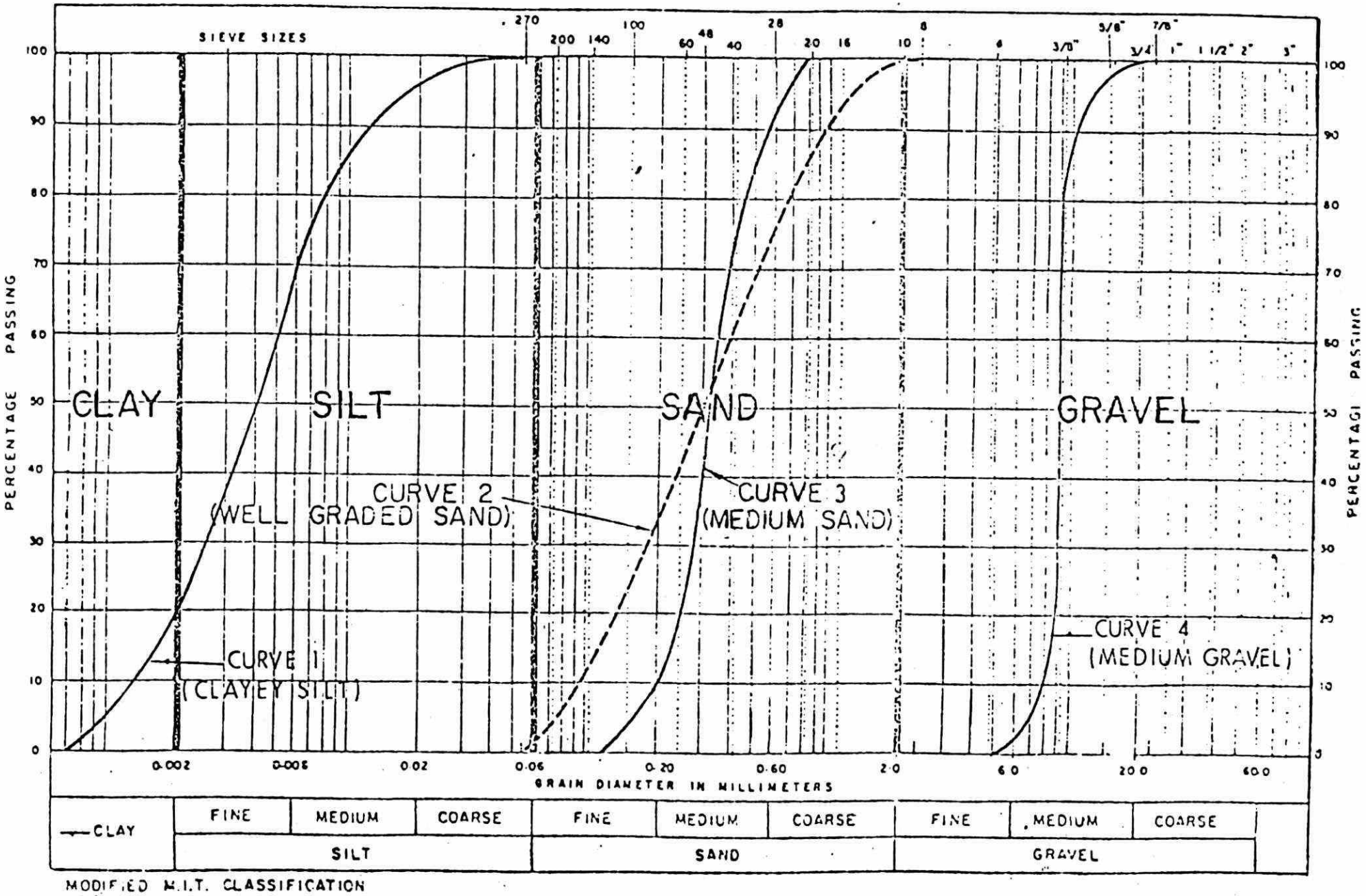
Earth Manual: US Department of the Interior

Field Identification Procedures (Excluding particles larger than 3 inches and basing fractions on estimated weights)				Group Symbols [1]	Typical Names	Information Required For Describing Soils	Laboratory Classification Criteria
COARSE GRAINED SOILS More than half of material is larger than No.200 sieve size [2] (The smallest particle visible to the naked eye)	Gravels (More than half of coarse fraction is larger than No.4 sieve size but smaller than 3")	Clean Gravels (Little or no fines)	Wide range in grain size and substantial amounts of all intermediate particle sizes	GW	Well graded gravels, gravel-sand mixtures, little or no fines	Give typical name, indicate approximate percentages of sand and gravel, max. size, angularity, surface condition, and hardness of the coarse grains, local or geologic name and other pertinent descriptive information, and symbol in parentheses. For undisturbed soils add information on stratification, degree of compactness, cementation, moisture conditions and drainage characteristics. Example:- Silty sand, gravelly, about 20% hard, angular gravel particles 1/2 in. maximum size; rounded and subangular sand grains coarse to fine, about 15% nonplastic fines with low dry strength; well compacted and moist in place; alluvial sand. (SM)	Use grain size curve in identifying the fractions as given under field identification. Determine percentages of gravel and sand from grain size curve. Depending on percentage of fines (fraction smaller than No.200 sieve size) coarse grained soils are classified as follows:- Less than 5% GW, GP, SW, SP. More than 12% GM, GC, SM, SC. 5% to 12% Borderline cases requiring use of dual symbols. $C_U = \frac{D_{60}}{D_{10}}$ greater than 4; $C_C = \frac{(D_{30})^2}{D_{10} \times D_{60}}$ Between one and 3. Not meeting all gradation requirements for G.W. Atterberg limits below "A" line, or PI less than 4. Above "A" line with PI between 4 and 7 are borderline cases requiring use of dual symbols. Atterberg limits above "A" line with PI greater than 7. $C_U = \frac{D_{60}}{D_{10}}$ greater than 6; $C_C = \frac{(D_{30})^2}{D_{10} \times D_{60}}$ Between one and 3. Not meeting all gradation requirements for S W. Atterberg limits below "A" line, or PI less than 4. Above "A" line with PI between 4 and 7 are borderline cases requiring use of dual symbols. Atterberg limits above "A" line with PI greater than 7
		Gravels with Fines (Appreciable amount of fines)	Non-plastic fines (for identification procedures see ML below)	GM	Silty gravels, poorly graded gravel-sand-silt mixtures.		
			Plastic fines (for identification procedures see CL below)	GC	Clayey gravels, poorly graded gravel-sand-clay mixtures.		
	Sands (More than half of coarse fraction is smaller than No.4 sieve size but larger than the No.200 sieve size (for visual classifications the 1/4" may be used as equivalent of No.4)	Clean Sands (Little or no fines)	Wide range in grain sizes and substantial amounts of all intermediate particle sizes	SW	Well graded sands, gravelly sands, little or no fines.		
			Predominantly one size or a range of sizes with some intermediate sizes missing	SP	Poorly graded sands, gravelly sands, little or no fines.		
		Sands with Fines (Appreciable amount of fines)	Non-plastic fines (for identification procedures see ML below)	SM	Silty sands, poorly graded sand-silt mixtures		
			Plastic fines (for identification procedures see CL below)	SC	Clayey sands, poorly graded sand-clay mixtures		
FINE GRAINED SOILS More than half of material is smaller than No.200 sieve size (The No. 200 sieve is about the smallest particle visible to the naked eye)	IDENTIFICATION PROCEDURES ON FRACTION SMALLER THAN No.40 SIEVE SIZE						
	Silts and Clays Liquid limit less than 50	Dry Strength (Crushing Charac.)	Dilatancy (Reaction to Shaking)	Toughness (Consistency near Pl. limit)			
		None to slight	Quick to slow	None	ML	Inorganic silts and very fine sands, rock flour, silty or clayey fine sand with slight plasticity	Give typical name, indicate degree and character of plasticity, amount and maximum size of coarse grains; color in wet condition odor if any, local or geologic name, and other pertinent descriptive information, and symbol in parentheses. For undisturbed soils add information on structure, stratification, consistency in undisturbed and remolded states, moisture and drainage conditions. Example:- Clayey silt, brown, slightly plastic; small percentage of fine sand, numerous vertical root holes; firm and dry in place; loess; (ML)
		Medium to high	None to very slow	Medium	CL	Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays.	
		Slight to medium	Slow	Slight	OL	Organic silts and organic silt-clays of low plasticity	
	Silts and Clays Liquid limit greater than 50	Slight to medium	Slow to none	Slight to medium	MH	Inorganic silts, micaceous or diatomaceous fine sand or silty soils, elastic silts.	
		High to very high	none	High	CH	Inorganic clays of high plasticity, fat clays.	
		Medium to high	None to very slow	Slight to Medium	OH	Organic clays of medium to high plasticity	
Highly Organic Soils		Readily identified by color, odor, spongy feel and frequently by fibrous texture			Pt	Peat and other highly organic soils.	



1. Boundary classifications:-Soil possessing characteristics of two groups are designated by combinations of group symbols. For example GW-GC, well graded gravel-sand mixture with clay binder.

MECHANICAL ANALYSIS



MODIFIED M.I.T. CLASSIFICATION

MECHANICAL ANALYSIS

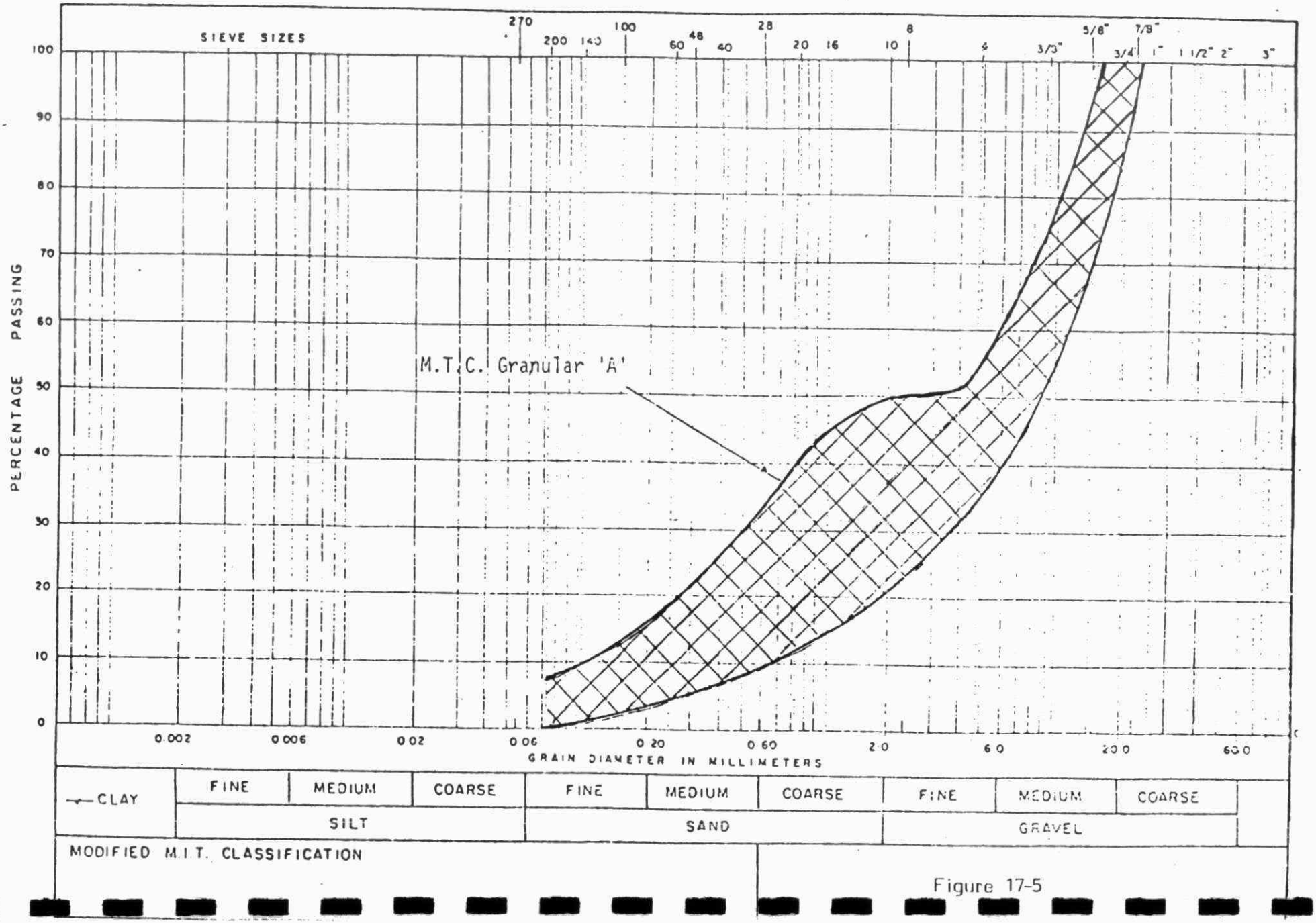


Figure 17-5

MECHANICAL ANALYSIS

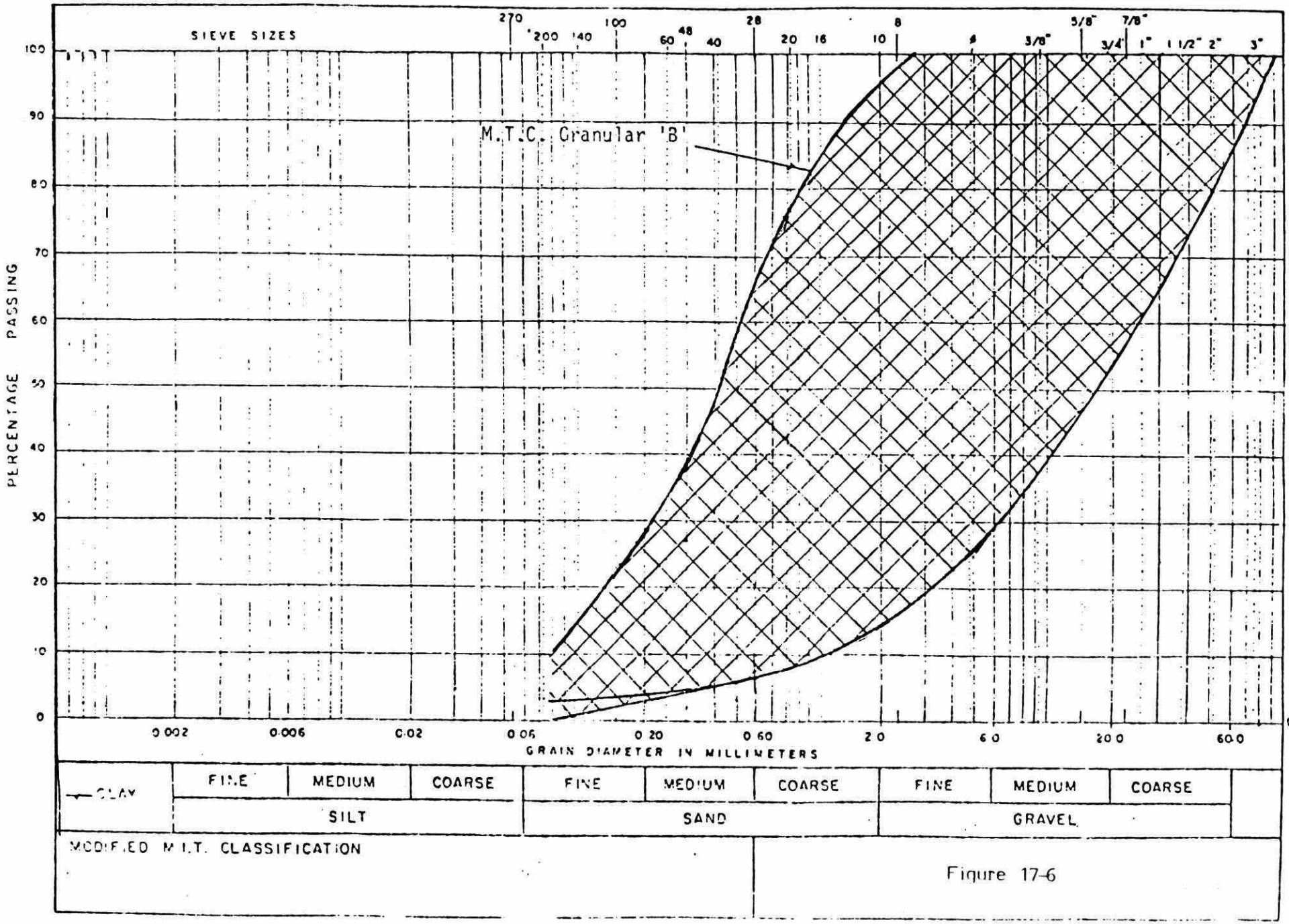
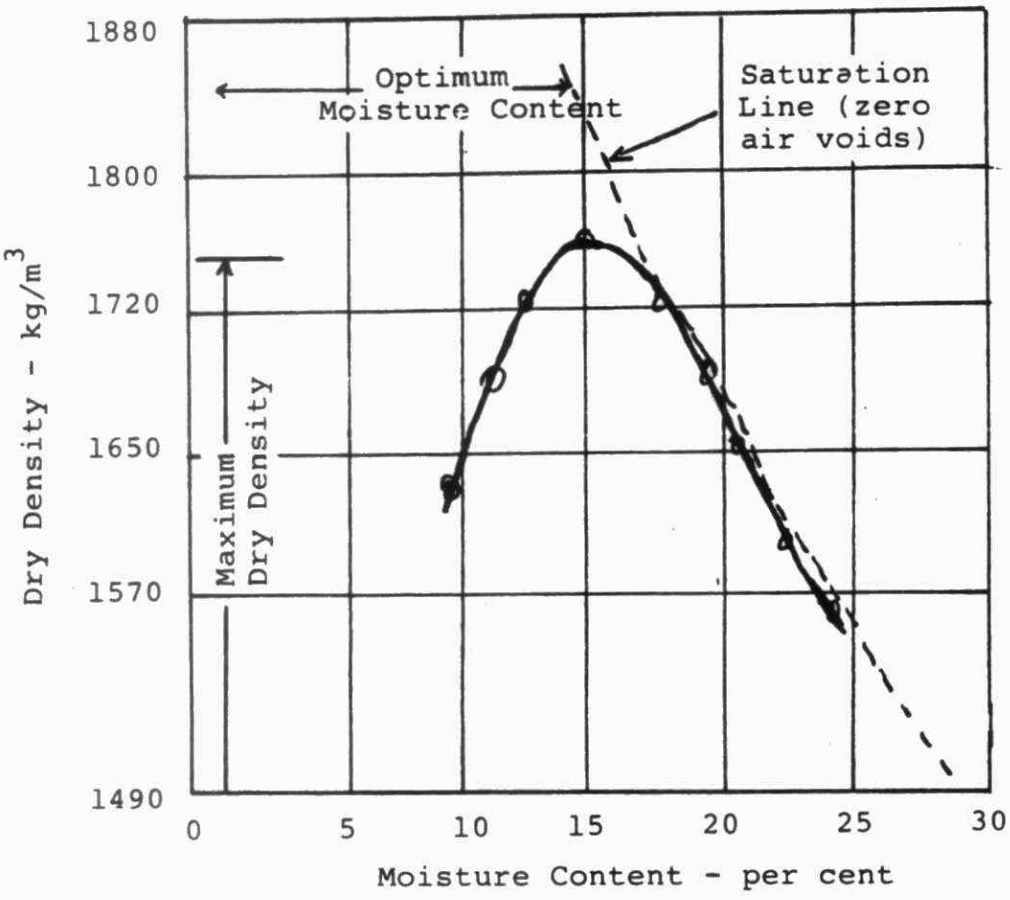
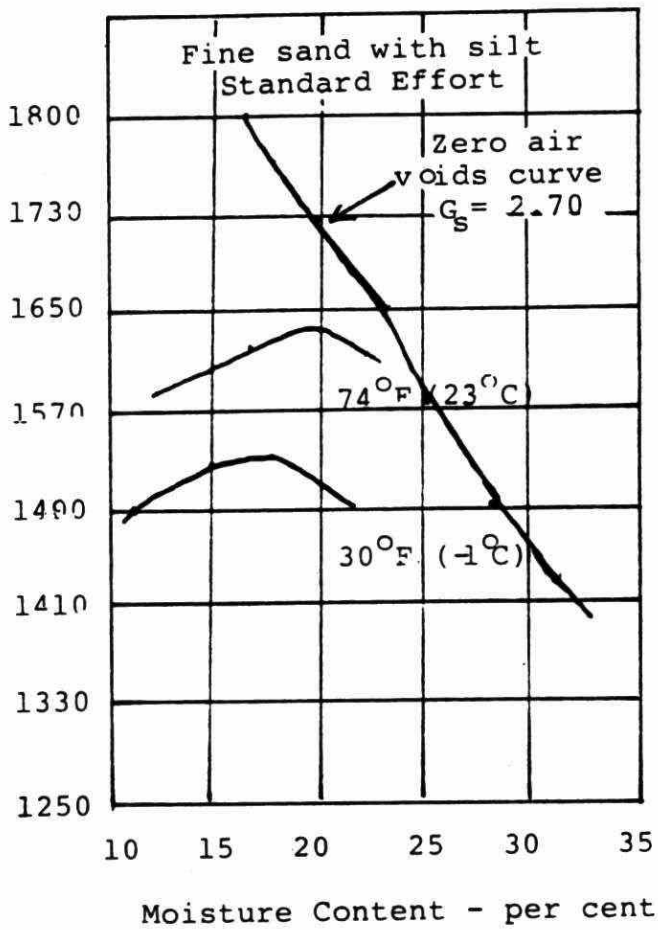


Figure 17-6

RELATIONSHIP BETWEEN DRY DENSITY AND MOISTURE CONTENT FOR CONSTANT AMOUNT OF COMPACTION

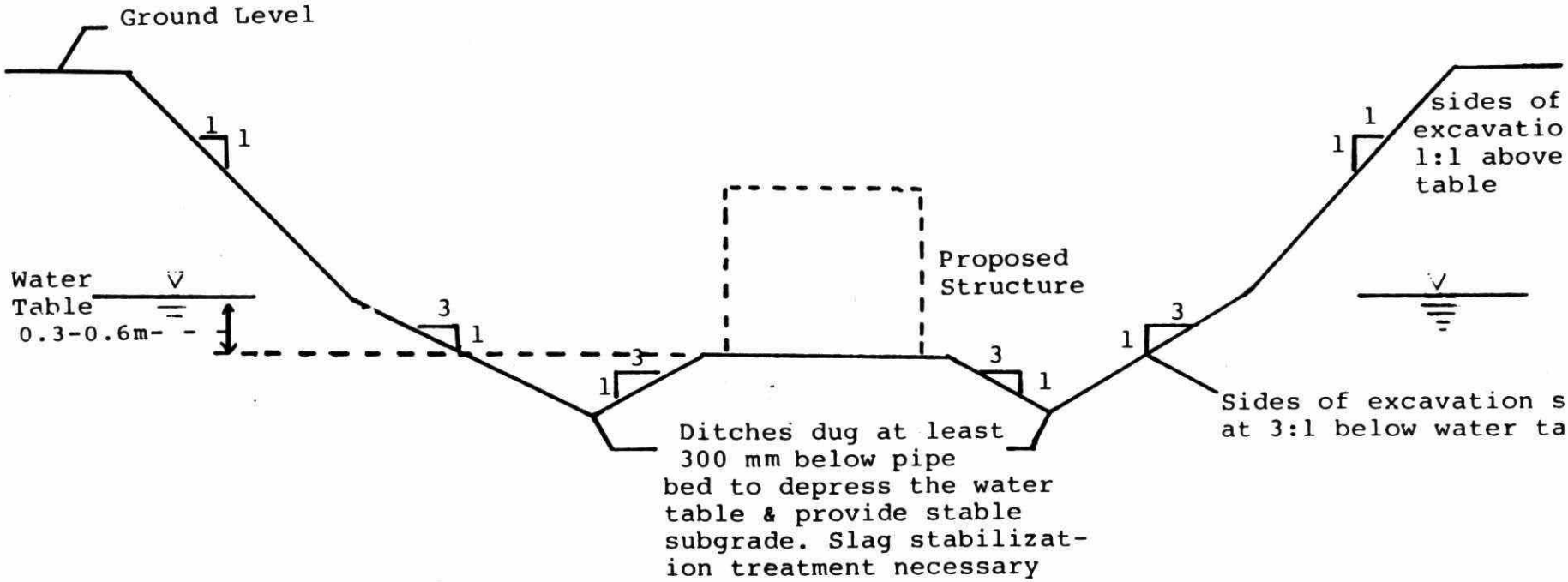


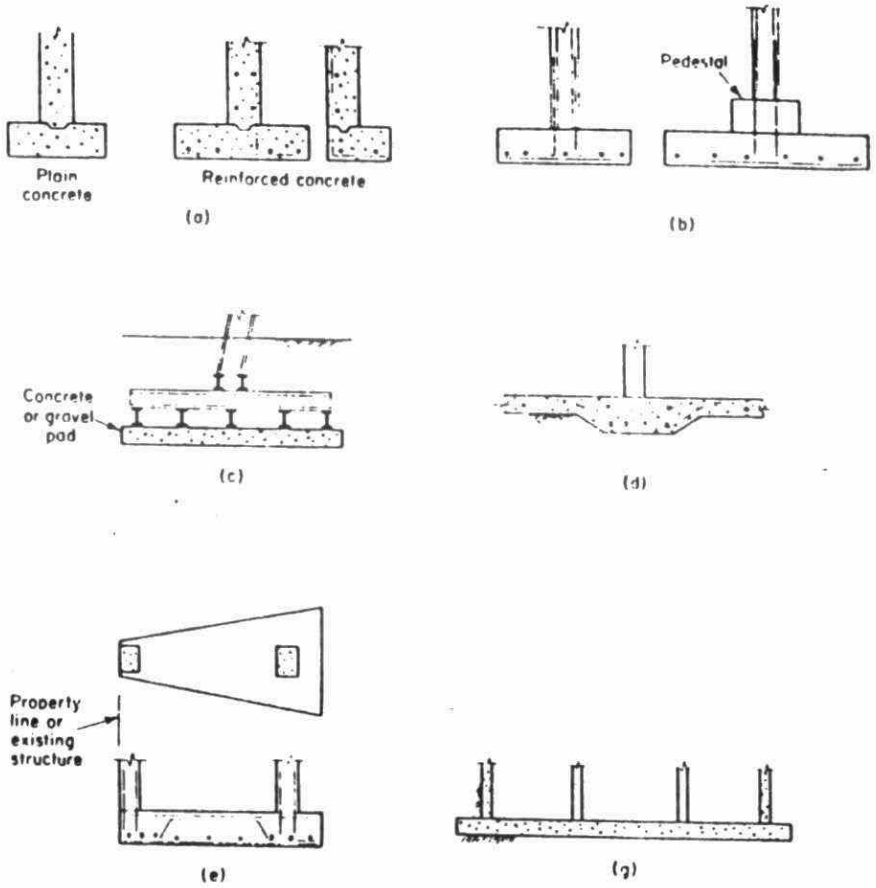
EFFECT OF FREEZING TEMPERATURES ON COMPACTION OF A SILTY FINE SAND



DESCRIPTION	OPTIMUM M/C %	STANDARD PROCTOR DRY DENSITY 100 kg/m^3
Granular 'A'	5 to 9	20.0 to 21.8
Granular 'B'	8 to 13	18.5 to 20.4
Silty Sand & Gravel	8	19.4
Fine to Medium Sand	9 to 14	17.4 to 18.2
Coarse Sand	5	17.9
Fine Sand	14	16.5
Silty Sand	12	17.7
Silt	15 to 18	16.0 to 16.6
Silty Clay	22	15.8
High Plastic Clay	28	14.4

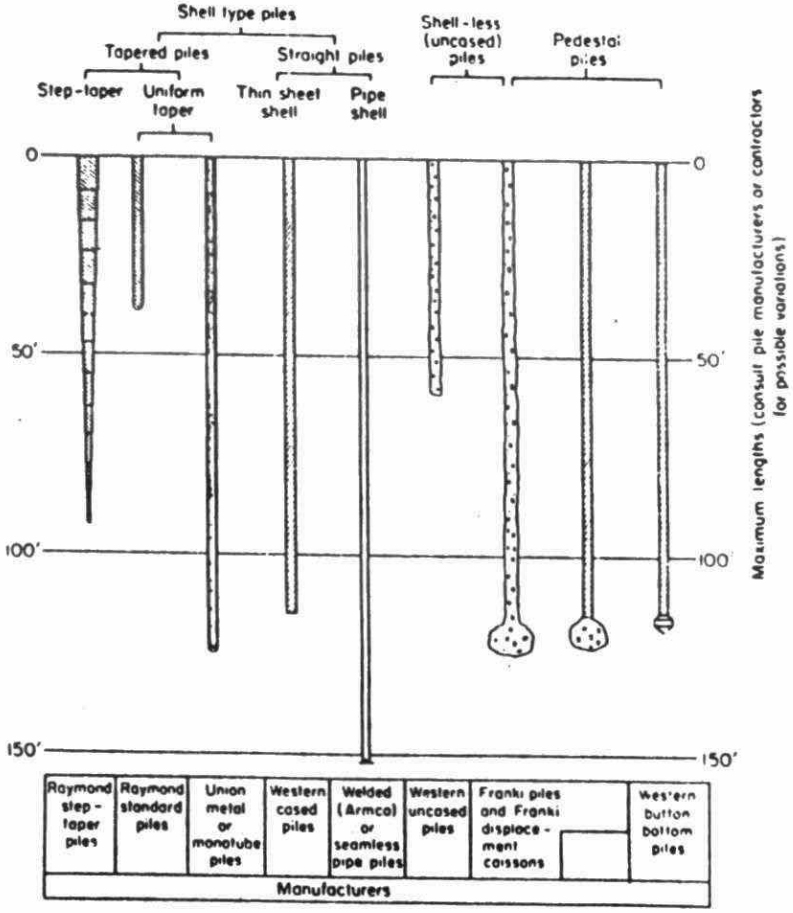
SKETCH OF RECOMMENDED DRAINAGE PROVISIONS FOR UNBRACED EXCAVATIONS IN WET SAND BELOW THE WATER TABLE





Common types of footings: (a) wall or continuous footings; (b) spread footing (square, rectangular, or round) uses pedestal to (i) reduce thickness of footing, (ii) develop bond strength of dowels; (c) steel grillage (steel beams usually galvanized or encased in concrete) is largely used in transmission tower foundations or under heavy column loads; (d) monolithic footing is used for watertight basement and for resisting uplift pressure; (e) combined footing; (g) raft or mat foundation is used in soils with low bearing capacity

TYPICAL SHALLOW FOUNDATIONS



Common types of cast-in-place piles.

TYPICAL DEEP FOUNDATIONS

SUBJECT:

CONSTRUCTION OF SEWERS WATERMAINS AND ASSOCIATED FACILITIES

TOPIC: 11

PRACTICAL ASPECTS OF DEWATERING

OBJECTIVES:

The trainee will be able to:

1. Identify the various methods employed for dewatering of excavations.

2. Understand the installation and the function of each method.

3. Describe the factors governing the application, installation and operation of each method of dewatering.

PRACTICAL ASPECTS OF DEWATERING

INTRODUCTION

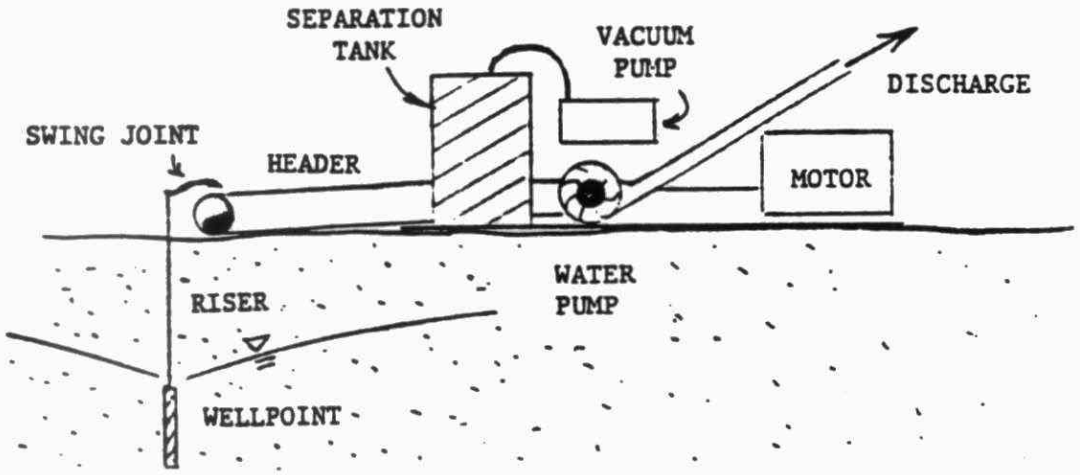
Dewatering or predrainage of construction sites is commonly carried out by vacuum wellpoints, deep wells or eductor wells. The function of each system is to efficiently remove only enough water from an aquifer to facilitate underground construction. The choice of dewatering method is a function of aquifer characteristics, depth or required drawdown, site geometry and, above all, cost. To describe in detail all the factors governing the application, installation and operation of a dewatering system would require many volumes of text. In the following pages of brief description of the mechanical aspects and practical considerations of each system will be given.

VACUUM WELLPOINTS

Vacuum wellpoints systems are commonly used to dewater excavations for trenches and building foundations, to reduce the hydrostatic load on sheet-piled cofferdams, control seepage through earth fill berms and dykes, and have recently been used to control ground water flow around waste disposal areas and remove spilled petroleum products flowing on the water table. A vacuum wellpoint system relies on atmospheric pressure to remove the water upward to the ground surface where it can be collected or discharged. Typically, a wellpoint system can achieve 6 metres of drawdown at the wellpoint line. Away from the header line, the drawdown varies exponentially with the distance to the header line and depends on aquifer characteristics. If there are no space restrictions to open cutting, greater depths of drawdown than 6 metres can be achieved by staging the wellpoints at different levels.

The basic components of a vacuum wellpoint system are shown on Figure 1.

Figure 1 Vacuum Wellpoint System



The wellpoint screen and riser pipe are placed in the ground and connected by means of a swing joint and valve to a common manifold called a "header pipe". The wellpoint pump includes a vacuum unit to create low air pressure in the header/wellpoint system and when atmospheric pressure forces water up to ground level, it is collected in a separation tank for discharge by the pump. Valves are attached in line to control the flow of water and air through the system. The wellpoint pump unit can be powered by diesel, gasoline, or propane-fuelled internal combustion engines or electric motors.

In the past, the components of the system have been made of steel, however, in the last twenty years or so, plastic wellpoints, riser and header, and aluminum header have been successfully introduced. While steel systems are more durable and hence used extensively in the presence of construction traffic, lightweight plastic or aluminum are preferable where the equipment must be moved ahead frequently, as in trenching operations. Plastic components are virtually inert and agents are found in the groundwater. Because of their lower capital cost, plastic components are more suitable in situations where parts of the wellpoint system must be left on site permanently. Plastic wellpoints are ideal when wellpoint length must be varied to accommodate stratigraphic changes on site. Wellpoint headers are commonly 150mm or 200mm in diameter although as small as 100mm and as large as 600mm have been used. The size of the header is chosen to minimize friction losses in the system. Since the volume of flow increases along the header line in the direction of the pump, it is not uncommon to find small header sizes, usually at the extremities of the system, and larger diameters near the pump. In order to minimize friction losses along the header lines, it is always preferable to install the pump in the middle of the header system. It is best to have the pump located at a slightly higher elevation than the header lines so that air, which accumulates in the header pipe, will easily flow into the pump. If the header line has high points, the pump discharge will surge as a result of large volumes of entrapped air breaking out of high points in the header. This surging can be damaging to the pump.

The pump is sized to handle flow which is larger than the anticipated steady state flow. In unconfined aquifers, the volume of flow decreases significantly with time as the groundwater comes out of storage. In this situation, the pump must be over-sized or else the time required for drawdown will be considerably greater. The question "how long until the water is drawn down" is often asked. The answer is not as simple as the question. Since there is usually no pump test data available on a given project, it is impossible to predict the answer accurately. However, when dewatering a confined aquifer, drawdown is usually achieved in about two days. Unconfined aquifers require longer periods of time to dewater.

Various installation methods are available and the most suitable method for a particular project is chosen on the basis of the ground conditions, the depth of the wellscreen and the need for a sand pack around the screen.

The SELF-JET method of installation is the least costly method of all as it simply involves lowering the wellpoint into a hole which is formed by water spraying out of the tip of the wellpoint. This method is ideally suited where the ground is relatively loose or erodible. Where filter sand is required, the self-jet method is limited to conditions where the jetted hole remains open to permit installation of sand. Filter sand is placed around the wellpoint screen by shovelling it into the annulus around the riser pipe. Care must be taken to reduce the volume and velocity of flow out of the wellpoint so as to fall to the bottom of the wellpoint.

The SAND CASING method involves washing a 200mm to 300mm diameter casing into the ground to the depth of the wellpoint. Once the casing is placed in the ground, the top is removed, the wellpoint is lowered into the open casing, and filter sand is introduced around the wellpoint. The sand casing method relies solely on the washing action of the water and the weight of the casing to penetrate the ground and is therefore limited to easily erodible soils. The wash water returns between the casing and the ground thereby minimizing smear.

The JET AND CASING method is one of the most versatile methods of installing wellpoints. A casing, having a diameter of 300mm or more, is lowered into the ground through the hammering action of a jet tube which fits inside the casing. The jet is fitted with a heavy top so that the casing can be hammered into the ground in tough soils which are not easily erodible. The wash water in this method generally comes up inside the casing. The jet and casing method is the preferable method for installing eductor wells and has been used to depths in excess of 50 metres.

Where tough ground is encountered above the aquifer, it is often preferable to PRE-AUGER at the wellpoint locations to permit installation of the wellpoints by any of the above methods. For this purpose, a variety of augers ranging from caisson drilling rigs to fence post augers have been used. On some rare occasions casings have been VIBRATED down to the aquifer to permit insıallation of the wellscreens into a cased hole.

In ground where a high incidence of cobbles is found, the HOLE PUNCHER method is suitable for installing wellpoints within cobbles. The hole puncher is a heavily weighted, thick walled, thin diameter (150mm) jet tube with a bull noze through which water is pumped. Constriction at the tip of the hole puncher causes the high velocity and high volume of water to displace large sized particles. If required, the hole puncher can be connected to an air compressor to provide additional force to dislodge cobbles. With the hole puncher method, filter sand is usually not used although it can be placed around the wellscreen by lowering filter sand around the wellpoint as the hole puncher is withdrawn and water is discharged from the tip of the point.

The water pressure for the above jetting methods is provided by a high pressure, high volume pump having a discharge pressure of 10 to 15 atmospheres or more and discharge volumes of 2,000 litres per minute or greater. The water supply for jetting is usually taken from fire hydrants or local creeks and ponds. Where fire hydrants are used to feed a high pressure pump, it is preferable to collect the water in a surge tank to minimize the risk of contamination of the water supply. Where hydrant water is not available, a water tanker may be required to install the initial part of the system and at that point, the system can be activated to provide water for jetting activities. The volume of water required to install wellpoint screens can vary from as little as 200 litres for a self-jet wellpoint to many tens of thousands of litres for a deep installation using a jet and casing method.

DEEP WELLS

Deep wells are used to dewater ground where there is a large wetted depth below excavation level and pervious soils exist resulting in high well yields. Vacuum can be applied to the wells to increase yields.

In principle, deep wells are the simplest method and certainly the oldest method of removing water from the ground. A deep well relies on gravity flow of the groundwater into the well where the water is removed by mechanical action. This mechanical action is usually imparted by a submersible turbine pump powered by an electric motor, located below the pump. In very high volume applications, vertical line shaft pumps with the motors at ground surface are used. Deep wells are unlimited in depth capacity, a factor which only depends on pump characteristics. Typical well diameters vary from 150mm to 600mm and pump capacities from 200 litres per minute (L/m) to many thousands of litres per minute.

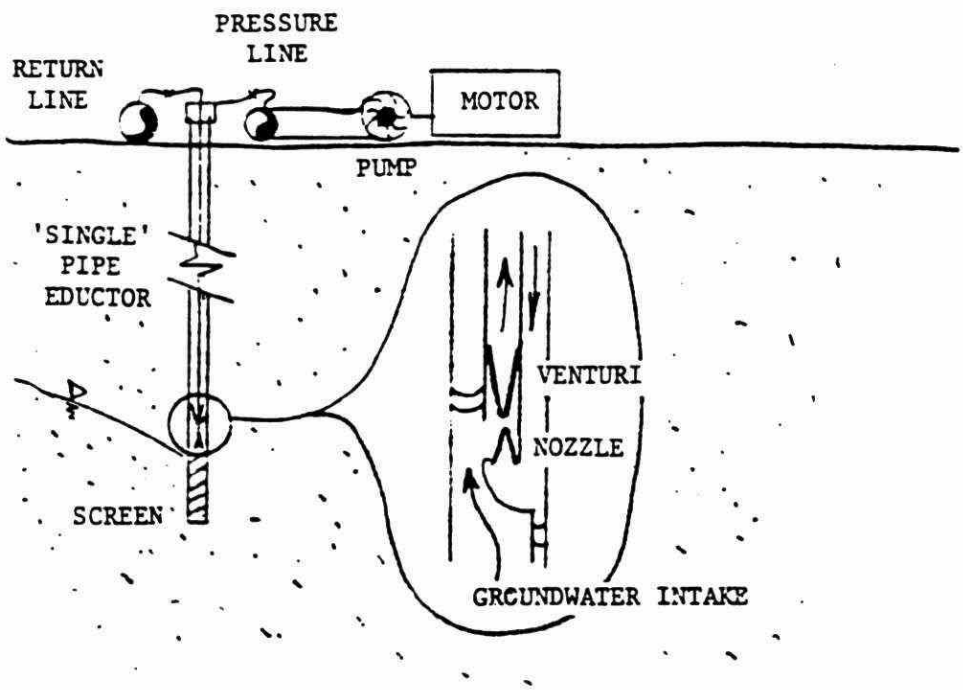
The holes in the wellscreen can be formed by sawn slots, punched and perforated indentations or the gap between wire wrapping. Wellscreens are made of plastic, fibreglass, steel, stainless steel or even wood. Typical opening sizes vary from 0.15mm to 0.75mm.

Deep wells are usually drilled into place using rotary, cable tool or bucket auger rigs. In suitable ground conditions they can be jetted into place at a considerably lower cost. Once in place, the soil around the screen must be developed to maximize flow in, yet keep sand out of the well. This is accomplished by surging the well through the intermittent movement of water through the wellscreen by gravity or pressure flow. An alternative to this self development, which work best in well graded soils, is the installation of a sand or gravel pack around the screen. Filter packs should be at least 75mm thick and require more complex installation procedures. Once developed, the pump is inserted to the top of the wellscreen. Deep well submersible turbine pumps have electric motors below the intake so that water flowing up from the wellscreen cools the motor. Discharge from the pumps is by steel pipe to the top of the well. In order to ensure that the pump does not run dry, the water level in the well is controlled by reducing the pump discharge, or by float activated switches which turn the pump on or off, or by electronically timed and controlled pump action. In dewatering applications, the pump is over-sized to ensure maximum drawdown and yield from the well. Deep well systems have a high unit cost but are relatively efficient since the work done is directly related to the energy consumption of the pump and friction losses are minimal.

EDUCTOR SYSTEMS

Eductor or ejector systems have the ability to lift water from great depths and are more suitable in fine grained soils where the depth of dewatering is greater than 6m. They are commonly used in applications where work space is limited, such as around sheet pile walls, for tunnel and mine shafts, behind soldier beam and lagging shoring systems, and along tunnel headings. Eductor systems can either be of a single or double pipe design. In the single pipe system (see Figure 2) a large diameter pipe forms the well casing and a smaller inner pipe forms the return line. Water is pumped under high pressure along the annulus between the two pipes and is forced through the nozzle and venturi. As a result, a low pressure is formed around the nozzle and this can be used to draw water through the well and up into the return line.

Figure 2



Typical operating pressures are in the range of 7 to 10 atmospheres on the pressure side. Eductor systems are powered by high pressure pumps which can be either electrically or diesel driven. Two header pipes are placed along the ground surface, on being the pressure line and the other the return line. Friction losses are a major factor in the engineering of eductor systems. due partly to the narrow opening between the annulus of the outer and inner pipe, friction losses are very high and eductor pipes must be sized accordingly. The most common eductor unit available at the present time consists of a 50 mm diameter outer casing with a 32 mm inner pipe. Nozzle and venturi sizes can be changed to accommodate expected flow and head conditions.

One of the limitation of an eductor system is that each eductor point is limited in the quantity of water which can be pumped, usually in the range of 20 to 60 litres per minute. One advantage of the eductor system is that the vacuum is developed right at the wellscreen and, with a properly installed clay seal above the sand pack around the point, a considerable vacuum can be transmitted to the soil.

A rather common occurrence with eductor system is the build up of algae or iron bacteria in the system. The algae accumulate near the nozzle and venturi and greatly increases the friction loss within the system. The simplest way to remove the algae is through chlorination of the water being pumped into the system. Iron bacteria can be either treated by chlorination or by the application of acid, depending on the nature of the bacteria.

PRACTICAL CONSIDERATIONS

When the water level is below the top of a wellpoint screen, air enters the system and reduces the ability of a vacuum wellpoint system to lift water. To remedy this situation the system must be "tuned" by turning down valves of the individual wellpoints so as to reduce the volume of air entering the system to the point so that the water level will stabilize just about the top of the wellscreen. The art of tuning requires careful observation and adjustment of the individual wellpoints, the vacuum level on the system, and the discharge volume. The entry of air into a vacuum wellpoint system is most common in unconfined sand aquifers. In the case of a confined aquifer, air entry can be minimized by the placement of a clay seal at the ground surface to prevent air from moving down the wellpoint screen along the sand column around the riser pipe. The entry of air into an eductor or deep well system does not reduce the system's ability to remove water.

Figure 3

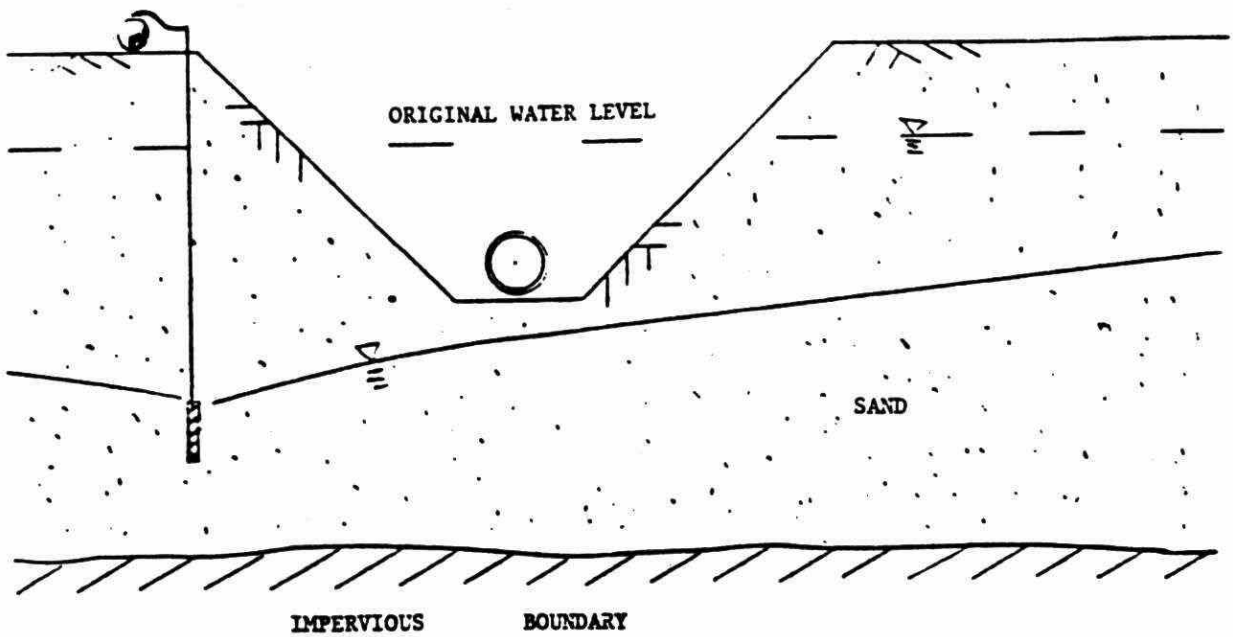
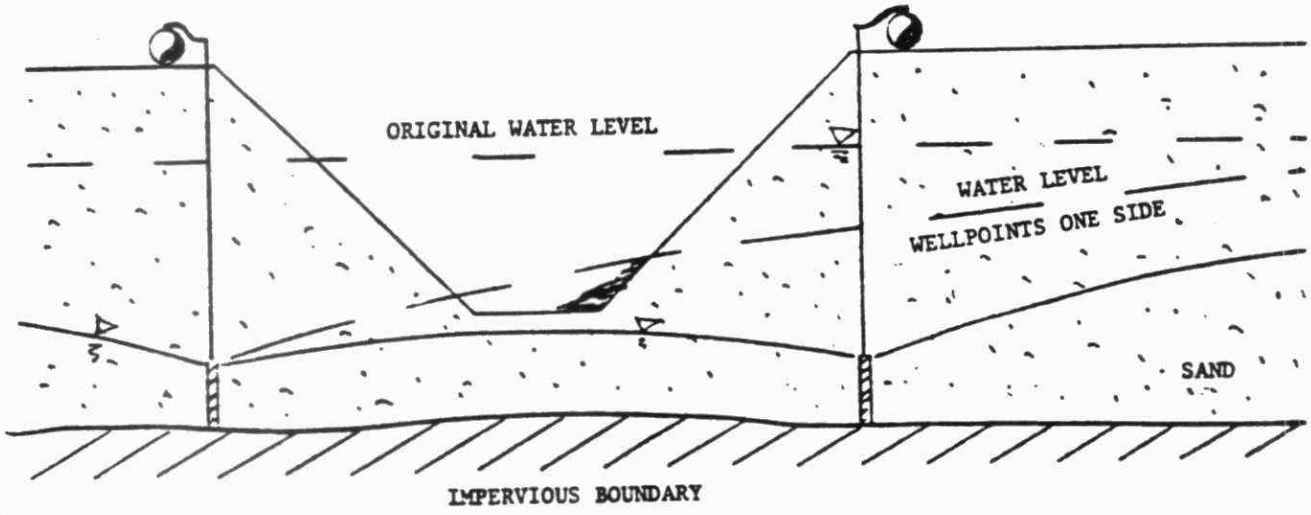


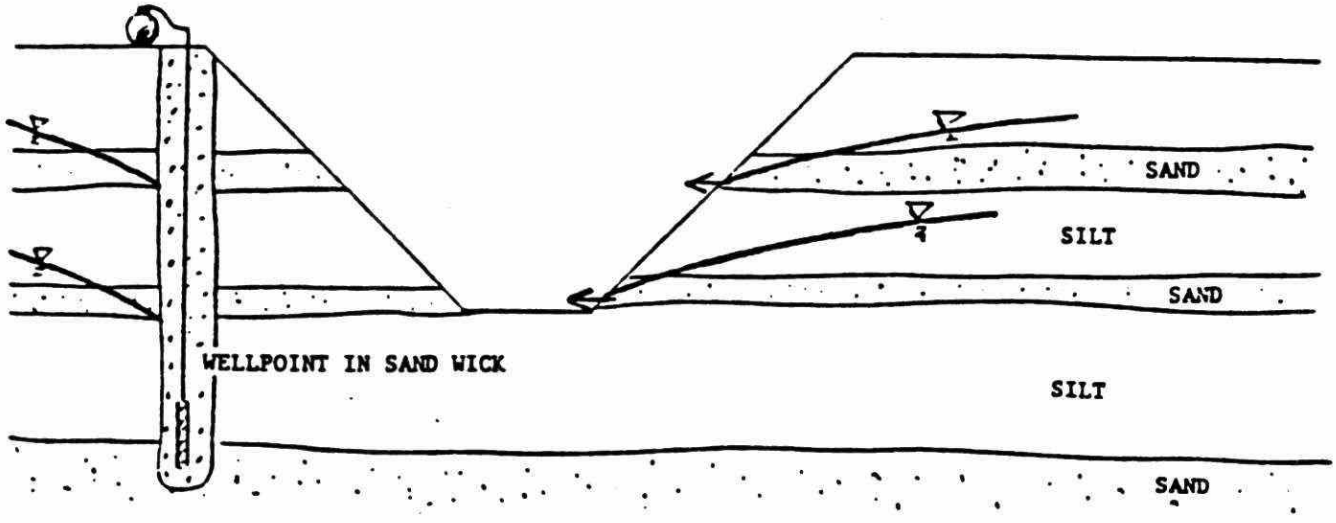
Figure 3 shows an idealized dewatering system lowering the water table to just below excavation level in a trench. The depth of the dewatering system and its capacity is designed so that the water table is at sufficient depth below the excavation at the opposite side of the trench to ensure that no ground water enters the excavation. However, when there is an impervious boundary at a relatively shallow depth below the base of the excavation, as in figure 4, a single line dewatering system may not lower the water table to below the excavation level at the far side of the excavation. In this case, the dewatering system must be installed on both sides of the trench. Shorter length wellpoints are also used in this application to achieve drawdown as close as possible to the impervious stratum.

Figure 4



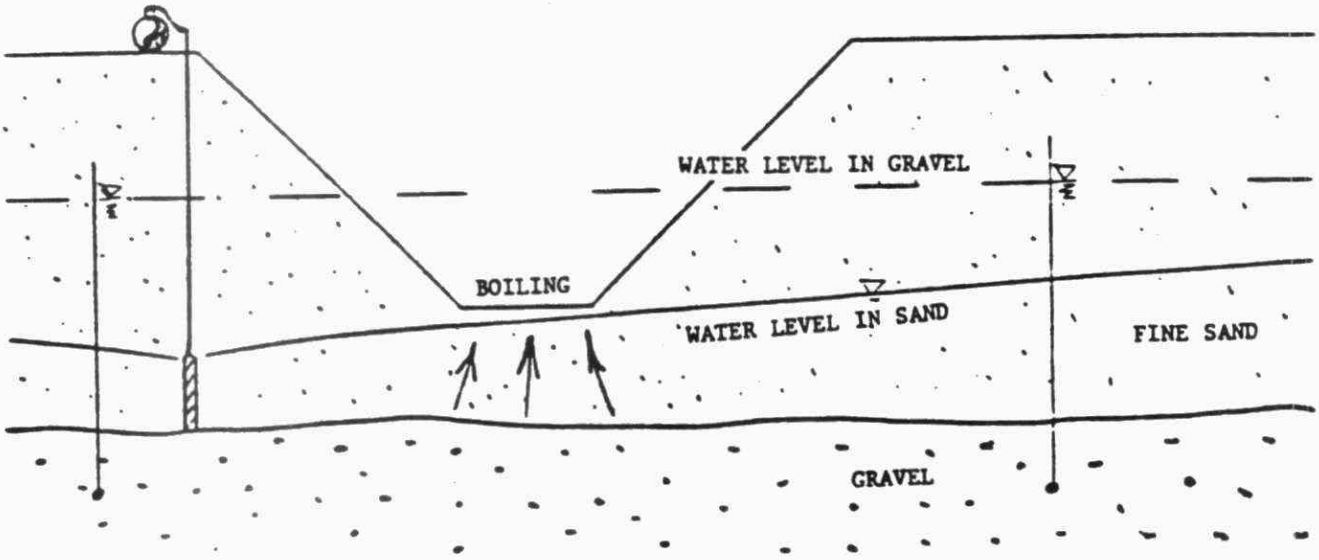
Similarly, where stratified conditions exist, as shown in Figure 5, groundwater may enter the excavation on the opposite side of the trench from a confined pervious aquifer where the dewatering system is installed only one side of the trench. In this case, also, dewatering systems would have to be installed on both sides of the trench and, in addition, a sand wick should be provided to drain the confined layers above the wellscreen.

Figure 5



Where an aquifer of significantly higher permeability exists below an aquifer of lower permeability (see Figure 6) the placement of wellscreens only within the finer soils as shown, may not be adequate. Vertical seepage from the gravel layer below can cause instability of the base of the excavation if the water level in the gravel is not controlled. The dewatering system would be most effective if it pumps water from the aquifer of higher permeability.

Figure 6



Appropriate wellpoint spacings are a function of the depth of required drawdown, soil permeability, the presence of impermeable layers and other factors. For example, wellpoints can be spaced far apart in a homogeneous sand aquifer where a small drawdown is required, however, in stratified soil, closer spacings would be required to intercept water perched on the upper layers. Typical wellpoint spacings vary from 1 to 4 metres. One controlling factor in the design of wellpoint spacings, is that the total flow into the area must not exceed the accumulative capacity of the individual wellpoints.

To form a sand wick around the wellpoint screen and riser, a filter sand is chosen which has a uniform gradation and is as coarse as possible to maximize permeability. The filter sand must however be fine enough to prevent migration of the native soil into the filter. for conventional wellpoint and eductor screens, a suitable filter sand consists of a coarse to medium sand with little or no gravel or fine sand. Unfortunately, the availability of this material is limited and a somewhat more well-graded material known as concrete sand is often used.

COST

The cost of dewatering varies widely depending on the method, the installation procedure, the amount of equipment on the job, and the duration. Most of the cost lies in the initial installation of the system and is typically more than 75% of the total cost. The operating costs of the system relies significantly on the method of powering the system. The most economical and reliable way to power a dewatering system is by means of electrically-driven pumps. Electrical energy consumption costs are about one-half to one-third of that of comparable diesel-powered systems. Costs cannot be related to volume of water pumped, the depth of drawdown or the volume of excavation, however, the size of the system and duration of use are loosely related to cost. As a rule of thumb, an eductor system is approximately triple the cost of a comparable length of vacuum wellpoint system. The applications of the two systems are however very different.

With electrically activated dewatering systems, diesel-powered standby generators are often provided which automatically engage upon failure of the main power supply. In addition, diesel-powered standby pumps are also provided in case of electrical or main pump failure. The provision of these items may add several thousands of dollars per month to the total cost of the system.

Dewatering work is often carried out on a subcontract basis where the subcontractor provides the equipment, installs it and removes it. Maintenance of the system is usually simple and therefore carried out by the general contractor who is always on site. In order to keep cost "in house", many contractors prefer to rent the equipment and install it with their own forces under the supervision of a technical specialist provided by the dewatering contractor/specialist.

RESPONSIBILITY

Given adequate information on the ground conditions at a site, the general contractor and his dewatering contractor are usually given responsibility for deciding on the need for, and the method of, dewatering. From a dewatering contractor's viewpoint, "adequate" information includes carefully detailed bore hole logs noting stratification, where groundwater was encountered during drilling, the water level in piezometers sealed into the specific water bearing stratum, and grain size distribution curves of the water bearing soils. From these items, which are generally provided in all geotechnical reports, quantity of flow and drawdown can be determined. Geotechnical consultants should recognize the severity of a dewatering problem and, if it is considerable, a pump test may be warranted. We should realize that, while pump tests are expensive, the well used for the test can usually be incorporated in the actual dewatering scheme and this, in addition to the value of more accurate information, will help to minimize risk from dewatering often arise from insufficient geotechnical information, particularly a lack of appreciation of the severity of the groundwater problem.

MONITORING OF LOCAL DOMESTIC WELLS

When there are private wells located near the construction site and dewatering is to be undertaken, such wells should be inspected and monitored prior to the start of any dewatering. Monitoring should continue at regular intervals throughout the construction and dewatering operation, to determine if there is any interference with the ground water source. A record should be kept of weather conditions, dewatering rates and well monitoring results, so these can be related to the conditions of the local domestic wells. If ground water interference is suspected, the inspector should immediately bring this matter to the attention of his supervisor and the contractor.

CONCLUSIONS

Groundwater can greatly influence the design and construction of a project. In some cases, the presence of groundwater makes the project technically impossible to complete without dewatering while in other cases the presence of groundwater simply makes construction activities slow and difficult. The use of groundwater control systems can therefore be a positive solution to the problem. The successful completion of any dewatering systems depends on appropriate engineering design, suitable installation and diligent observation of on-site conditions which may require modification to the design. While the basic principles of dewatering will not change, technical improvements to equipment must continue to be made to ensure that dewatering remains a cost competitive option available to all dealing with groundwater problems.

SUBJECT:

CONSTRUCTION OF SEWERS
WATERMAINS AND ASSOCIATED
FACILITIES

TOPIC: 12

BLASTING CONTROL
TECHNIQUE AND SAFETY

OBJECTIVES:

The trainee will be able to:

1. Be aware of governing legislation.
2. Be aware of some monitoring and control procedures.
3. Be aware of some explosive products.
4. Be aware of some initiation systems and procedures.
5. Be aware of safety procedures and some of the potential hazards.

BLASTING CONTROL

LEGISLATION

Federal

The supplying, hauling and storing of explosives and accessories is covered by Federal Statute under the rules and regulations of the Explosive Division, Department of Energy, Mines and Resources. In addition, under these regulations each user of explosives is required to maintain a strict record and control of the explosive stock in his possession and to be accountable for the disposition of each item of explosive product or accessory.

Provincial

The requirement that every precaution shall be taken in the handling, storage, transportation and use of explosives to protect persons on a project is covered by the Occupational Health and Safety Act and Regulations for Construction Projects, specifically Ontario Regulation 213/91 sections 196 to 206 inclusive of the Revised Statutes of Ontario, 1980, Chapter 321 as amended and Revised Regulations of Ontario, 1991, Regulation 213.

There is no provincial licensing requirement for blasters/powdermen in Ontario at the present time. However, under Ontario Regulation 213/91 section 196 made under the above mentioned Revised Statutes and Regulations of Ontario 1980, and similar requirements contained in Part VI, Section 116 to 147 of the Regulations for Mines and Mining Plants, it is required that a competent person shall be appointed to be in charge of and to personally supervise all blasting operations. The term "competent" is defined as "qualified by knowledge, training and experience to perform an assigned task". This regulation thus makes it the responsibility of employers and their project superintendents to ensure the competence of persons where prescribed by the Regulation.

Special note should be made of section 204 (2)(a), which prescribes that a drill hole shall not be drilled within 7.5 metres of a hole being loaded with or containing explosives. This restriction is subject to section 204, clause (3), (a) and (b), and clauses (4) and (5), which permit it to be waived, provided that,

(3) (a) A specification has been prepared in writing by a professional engineer describing the precautions to be taken to prevent the accidental detonation of the explosives in a loaded hole by the drilling of an adjacent hole, and

(b) the drilling is done in compliance with the specification.

(4) No hole is drilled within one metre of another hole containing explosives.

(5) The specification is signed and sealed by the professional engineer.

(6) A copy of the specification referred to in clause (3) (a) is kept at the project until the drilling and blasting operations to which it refers have been completed.

Municipal

A few individual municipal authorities, notably those of the cities of Ottawa and Hamilton, require that blasters/powdermen hold current local licenses while blasting within their jurisdiction. In general, however, municipal involvement is limited to the granting of blasting permits for specific projects or facilities.

Since August, 1978, an edition of the Model Municipal Noise Control By-Law, including restrictions on the permissible levels of noise and ground vibration from quarry blasting, has been made available to individual municipal authorities by the Ontario Ministry of the Environment, should they wish to adopt such a bylaw. The Ministry has adopted these levels as guidelines for long-term planning and the restrictions currently contained in the Model By-Law have thus become accepted standards for the control of noise and ground vibration effects from quarry blasting operations in Ontario. The specific limits contained in the August, 1978, Final Report of the Model By-Law will be dealt with in more detail in a later section, along with Ontario Provincial Standard Specification O.P.S.S. 515 which contains the current guidelines for construction blasting.

CONTROL PROCEDURES

Pre-Blast Surveys

Blasting is an effective and controllable engineering procedure, but it is also a spectacular and sometimes startling operation which can lead to the formation of false impressions in the mind of an observer, particularly if that observer is also a property owner.

Nowadays, in almost every project which involves any degree of blasting, it has become the rule rather than the exception for claims to be submitted by adjacent, and sometimes distant, property owners for damage allegedly caused by undesirable by products of the detonations. Very often, these claims are honestly made but nonetheless invalid.

Residents of adjacent properties, suspecting that if the blasting disturbs them personally it must also be capable of affecting their homes structurally, begin to look at the plaster, concrete and masonry components more closely than ever before. Not surprisingly, they find cracks which they have not previously noticed but which have quite possibly been in existence for some time.

It is clear, then, that human response alone is a most unreliable indicator of the damaging potential of ground vibrations, and that other more convincing evidence is required for evaluation of the validity of claims for blasting damage.

Ideally, therefore, before blasting is carried out on any construction site one should commence with a careful examination, preferably including still photographs or video photography, of all buildings liable to be within the range of perceptible vibration from the source, or at least all buildings adjacent to the perimeter of the site. In the case of a heavily built-up area, a minimum pre-survey range of 60m in any direction from a proposed blasting location is suggested.

If the pre-blast survey is carefully made, all existing cracks and defective conditions can be recorded and their number and location can be brought to the attention of the homeowner. Should a claim for damage subsequently arise, a post-operation check can establish which defects have occurred within the time frame of the blasting operations concerned.

Pre-blast surveys are not usually recommended in the case of quarry or mine blasting because these operations will probably extend over a much longer time duration, within which many defects are likely to occur as a matter of course due to natural, "non-blasting" causes such as settlement and seasonal humidity variation. Control of this type of blasting should rely even more on monitoring by instrumentation.

Monitoring and Criteria

When a commercial explosive is detonated, only part of the energy released is used up in accomplishing the intended purpose of the detonation, ie. generally, breaking rock to facilitate its removal in easily handled fragments. Hopefully, the proportion of energy so used will be large. But no matter how efficient the blast may be, some excess energy will always be available for dissipation as undesirable side effects, any or all of which, if uncontrolled, may threaten the structural integrity of adjacent vulnerable buildings, structures or services. These unwanted by-products of detonation are flying rock fragments, air concussion and ground vibrations.

Impact Identification

The environmental effects most often associated with blasting operations are ground vibration and air-blast. The intensity of ground vibration, an elastic effect, is measured in units of peak particle velocity, defined as the speed of excitation of particles in the ground resulting from vibratory motion. Peak particle velocity is measured in mm/sec.

Air blast is defined as a pressure wave travelling through the air, produced by the direct action of the explosive on air or by the indirect action of a confining material subjected to explosive loading. Noise is that portion of the spectrum of the air-blast lying in the audible range from 20-2000 hertz, while concussion is that portion lying below 20 hertz. For the purposes of control, air-blast is measured as decibels in the Linear or Unweighted mode (dBL).

The detonation of explosive product confined in a borehole produces a very rapid and dramatic volume change caused by conversion from a solid to a gaseous state, with the following effects:

i) The rock in the area immediately adjacent to the explosive product is crushed.

ii) As the energy from the detonation radiates outward from the borehole, the rock in front, i.e. toward the face, is fragmented and displaced while the rock behind is freshly cracked, or has existing cracks widened, by the explosives gases.

iii) Surplus energy from the detonations, i.e. the energy not used up in fragmentation, is dissipated in various forms, principally as ground vibration and air concussion or sound, attenuating rapidly with increasing distance.

The control of flyrock is largely a matter of common sense. If the shot is not obviously overloaded, if the holes are adequately stemmed and have sufficient collar, if the burden of rock at the free face and at the surface is sufficient and if the exposed surface area involved in the blast is adequately covered where necessary with good quality blasting mats or a sufficient depth of sand or other suitable material, then the flyrock problem will be controlled and the hazard therefrom greatly reduced if not entirely eliminated.

Some causes of flyrock are illustrated in Figure 12-1. Air concussion or blast overpressure effect is usually only a problem at relatively close quarters, and the most common hazard which it presents is window-glass breakage. This hazard too can be reduced by adequate stemming and the avoidance of overloading.

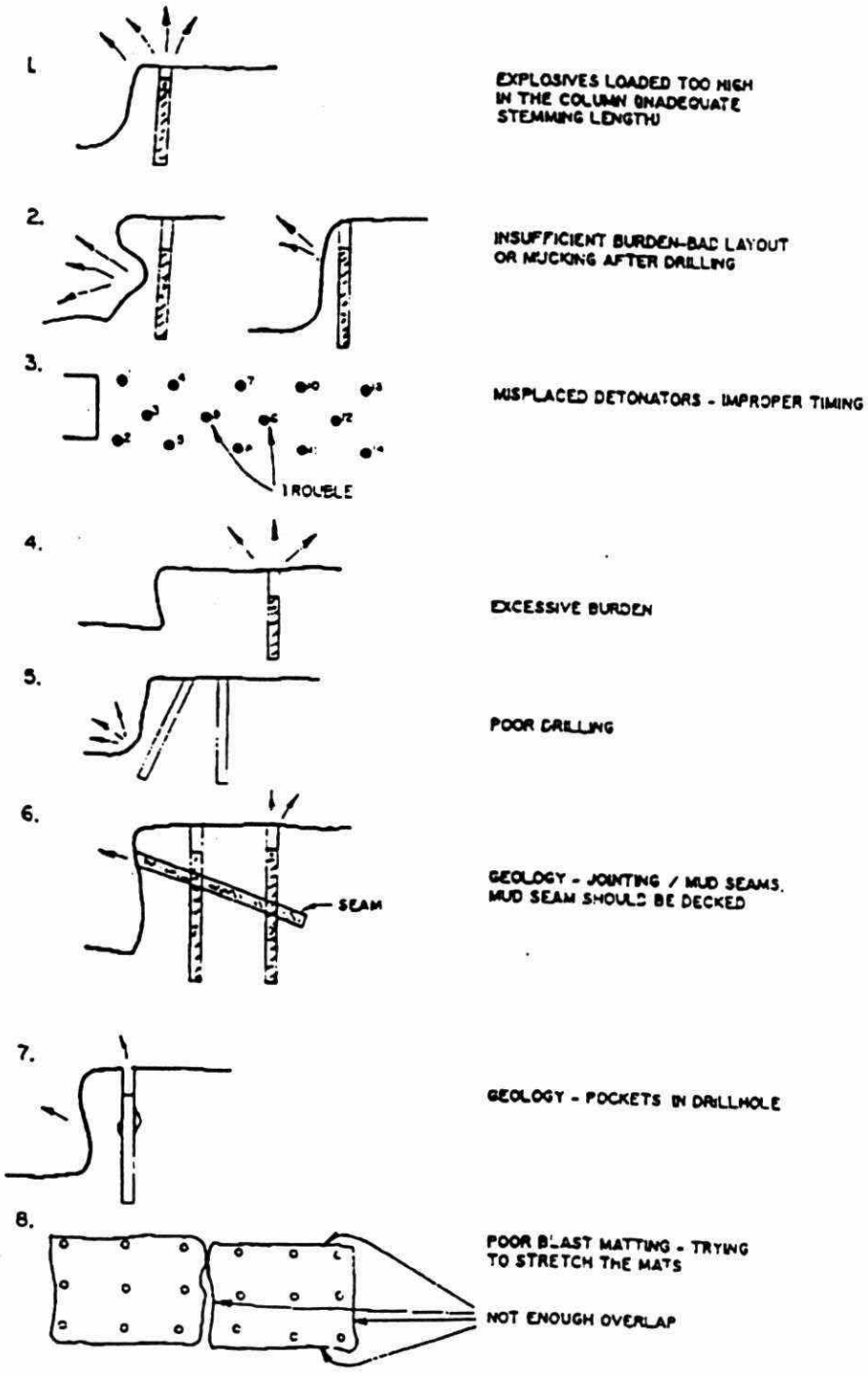
Figures 12-2 and 12-3 illustrate the effects of blast and sound overpressure. The damaging potential of ground vibration from blasting depends on the displacement and the frequency of the vibration. Neither of these two characteristics alone will damage a structure. The "particle velocity" of earthborn vibration is now generally accepted to be the best measure of damage potential, since it appears to combine displacement and frequency in the most significant manner.

It had been established from practical experiments by investigators in this country and abroad that particle velocities of between 71 and 81 millimetre per second in any one of three mutually perpendicular planes were normally required to reopen or extend old paster cracks or to produce new hairline cracks by ground vibration. With appropriate conservatism, the investigators agreed that a peak particle velocity level of 50 millimetres per second would be safe with regard to plaster cracks in residential-type structures. More recently however, it has been established that "threshold" or cosmetic damage can theoretically occur at velocity levels as low as 12.5 mm/sec at very low frequency levels, ie. within the 8-15 c.p.s. range generally associated with the natural or resonant frequency of structures.

Similarly, the limiting criterion for air over-pressure effect, to protect the structural integrity of the building closest to a blasting operation, had been originally established as that associated with an impact noise level of 140 decibels. However, noise complaints about blasting will increase rapidly from a level of about 112 decibels.

The Model Municipal Noise Control By-Law of August, 1978 referred to in an earlier section, establishes as a guideline a vibration cautionary limit of 10 mm/second and a peak vibration velocity limit of 12.5 mm/second. The latter limit is considered acceptable if the person in charge of a blasting operation carries out routine monitoring of the vibration velocity. The Model Municipal Noise Control By-Law peak vibration velocity limit is defined as the maximum particle velocity produced in any one plane.

Figure 12-1 Causes of Flyrock



BLAST OVERPRESSURE

Blast Overpressure is described as a compressional wave in air caused by:

a) an air pressure pulse caused by the direct rock displacement at the face or collar of a blasthole.

b) a rock pressure pulse caused by the vibrating ground

c) a gas release pulse caused by explosives gases escaping through fractures in the rock

d) a stemming release pulse caused by gas escaping from blown out stemming

Blast overpressure is usually low frequency although there can be a high frequency component (noise).

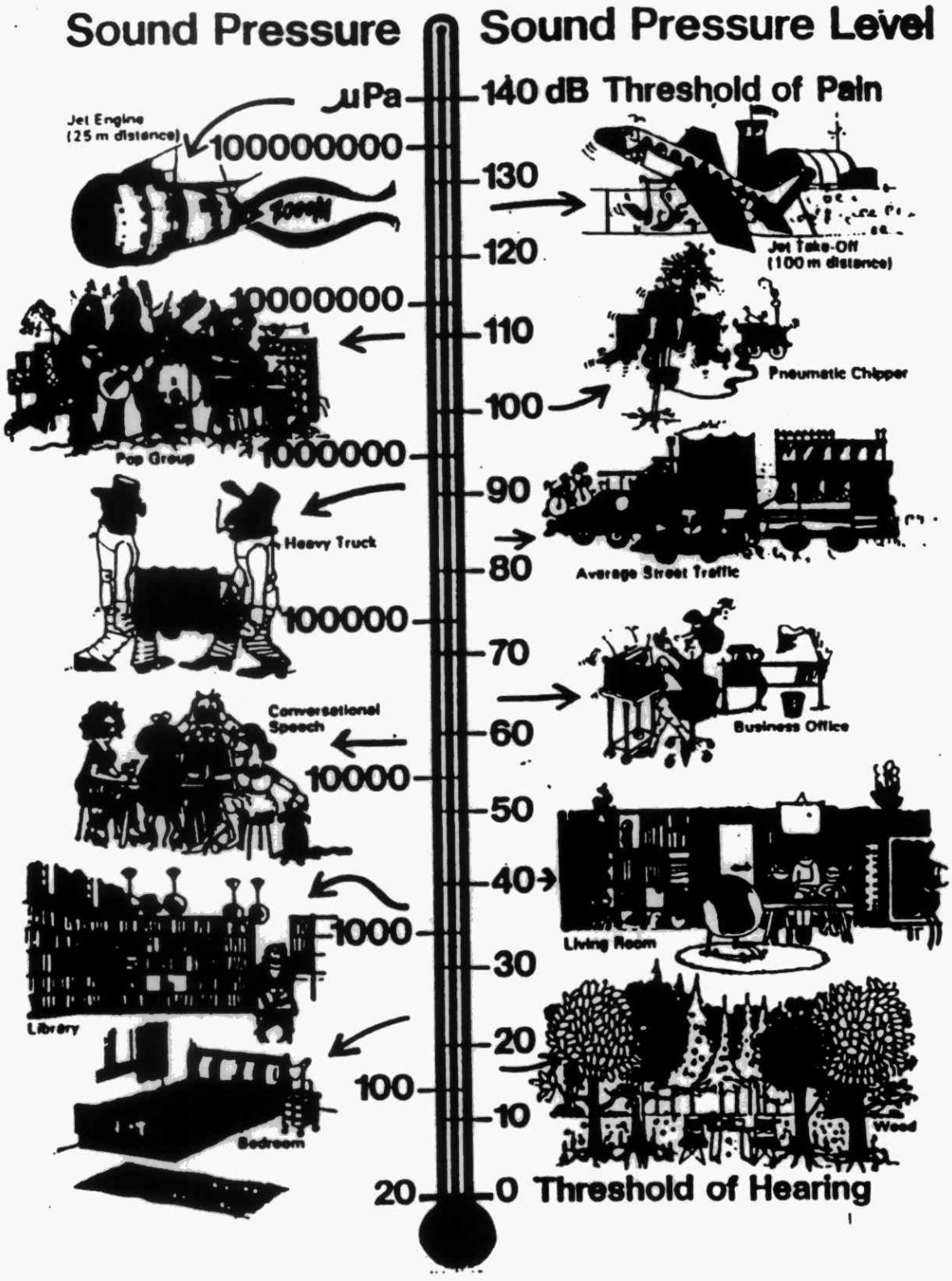
Overpressure				Air Blast Effect.
dBL			**.psi**	
181	-----	-----	3.0	Conventional structure severely damaged.
171	-----	-----	1.0	Most windows break.
161	-----	-----	0.3	
151	-----	-----	0.1	
141	-----	-----	0.03	Some large plate glass windows may break. USBM interim limit of allowable air blast 136 dBL.
131	-----	-----	0.01	

dBL = 20 log P/P_o where P_o is the reference pressure o. 0.0002 microbar.

The L signifies linear frequency response.

Figure 12-2 Overpressure is measured in dB, pascals or p.s.i.

Figure 12-3 Sound Pressure



Some of the effects of ground vibration are illustrated in Figures 12-4 and 12-5.

In place of the previously accepted limit of 140 dBL for impact noise level, the Model Municipal Noise Control By-Law of August, 1978 establishes a new Concussion-Cautionary Limit of 128 dBL. The latter limit is applied only if the person in charge of the blasting operation carries out routine monitoring of the over-pressure level. If the person in charge of the blasting operation does not carry out routine monitoring, the Concussion Cautionary Limit is 120 dBL.

It should be noted that the ground vibration and noise limitations described above apply only to the environmental side effects resulting from blasting operations in a mine or quarry. Under the more lenient terms of O.P.S.S. 515, construction blasters are permitted levels not exceeding 50 millimetres per second peak particle velocity and no limit for blast over-pressure has been established.

It will be seen then that the "transient" or construction blaster currently enjoys some advantage over the "resident" or quarry blaster in that a higher level of ground vibration intensity from relatively short-term construction blasting, typically at relatively high frequency, is considered tolerable compared to the low level restriction applied to quarries, where blasting activity may continue daily or seasonally for the lifetime of nearby residents, and where frequencies are sometimes lower, possibly within the resonant frequency range of some residences.

Figure 12-4 Structural Response at Different Scaled Distances from Pit Blasts

Figure 12-5 Anticipated Human Response at Different Scaled Distances from Pit Blasts

Effects on Water Supply

The potential effect of ground vibration from blasting on water supply is often raised during impact assessment or review. The United States Bureau of Mines, Department of the Interior, has funded an extensive study of this perceived problem, with the following conclusions:

1. When nearby blasting commences and blast-induced ground vibrations are relatively low (say, less than 1.0 in/sec or 25 mm/sec maximum particle velocity), the response of the well is limited to a slight variation in water level of the order of one or two-tenths of a foot either up or down. The effect is temporary and when the water well is tested by drawdown at a later date, the specific capacity is essentially unchanged.

2. Vibration measurements made at the surface and in the bottom of one of the observation wells indicate the vibration levels are always lower in the well.

3. All of the data collected in this study indicate that a limit of 2.0 in/sec (50 mm/sec) peak particle velocity is adequate to protect water wells from any significant damage. There is a possibility that temporary turbidity may be caused at lower levels from time to time but not at any constant threshold level.

With regard to the possible effects of blasting on natural aquifers in the bedrock strata, and hence on water supplies, it is also necessary to consider the non-elastic, or "plastic" effects produced very locally by each detonation.

Researchers in Manitoba, involved in blasting for hydro-electric dams in that Province found that formation of microcracks or the opening of discontinuities within bedrock required peak particle velocities in the range of 300 to 600 mm/sec. Experience indicates that in the limestone of Southern Ontario, such values need not be anticipated beyond a radius of 15 to 20 metres from any drillhole during detonation. The potential for damage to rock structure or natural aquifers by blasting activity may therefore be assumed to be restricted to an area within a very short distance from each blasthole.

Determining Attenuation

On any proposed construction site, it is advisable to begin blasting in a non-critical area, or alternatively, to detonate a series of conservatively light test blasts prior to the commencement of production blasting. These initial detonations should be monitored with two or more portable 3-component velocity-recording seismographs to establish the peak particle velocities produced and to determine the rate at which these tend to attenuate under the prevailing site conditions. An appropriately safe explosive load versus distance relationship can then be readily established and the maximum permissible explosive weight can be calculated for any critical distance from the initial production blast. By continuing to monitor the production blasting as it proceeds, on a continuous or intermittent basis, the explosive load versus distance relationship may be amended where necessary as site conditions change. Evidence of the continuing safe level of vibration intensity can be accumulated and stored in the form of permanent seismic recordings on disc or on paper, which are acceptable in court in the event of ensuing litigation.

Effects of Patterns and Procedures

As well as controlling the mechanics of fragmentation of the shot, much of the control of the potential side effects discussed earlier is dependent upon the particular blasting pattern and procedure chosen or designed for this purpose.

The more efficient a shot becomes, the less surplus energy is available for dissipation as side effects. The production of ground vibration in particular is largely governed by the distribution of explosive load by delay period detonation throughout the shot. Generally, it can be assumed that the intensity of ground vibration produced by the total blast will be no greater than that produced by the maximum load designed to be fired on any individual delay period.

In rare cases, an exception to this rule can occur in certain specific geological and groundwater conditions, where sympathetic propagation between holes may take place without the benefit of the designed delay period. Where such conditions are known to exist, or where sympathetic propagation has to be anticipated, explosive products which are not readily propagable, ie. not nitroglycerine-based, may be used if the explosive weight detonated instantaneously is liable to be critical.

Ground vibration intensities will invariably be greater per kg of explosive product for shots with very limited relief, such as sinking-cuts, or virtually no relief, such as pre-splitting, than for production blasting involving at least one vertical face as in trench blasting, or several faces as in many instances of highway construction. Production blasting in tunnel is an exception, since each "advance" involves the horizontal equivalent of a sinking-cut blast known as the "burn cut" and the ground vibration intensities will remain relatively high per kilogram of explosive product in comparison with other types of production blasting.

In situations which appear likely to be critical, ground vibration intensities from trench blasting can often be reduced appreciably by "mucking out" the previously blasted material up to the face of the next shot before firing, to provide a free face effect rather than a buffer.

Sewer and watermain contractors will sometimes use existing undisturbed overburden as a blasting mat by drilling through this material and loading the rock beneath. This procedure can be both economical and effective for the control of flyrock if the overburden is within an acceptable range of depth. Care must be exercised in these cases however, if it is considered that such limitation to normal relief during the blasting procedure might cause ground vibration intensities to rise to a critical level.

In view of the foregoing, the initial program of test monitoring of seismic effects referred to earlier should be carried out wherever possible under the most extreme conditions likely to be encountered for the production of a high intensity of ground vibration. These include sinking cuts, pre-shear blasts, buffered shots or detonation involving "in-situ" overburden.

CONTROL SPECIFICATIONS

There is often considerable variation in requirements concerning blasting control from project to project, even when the projects themselves appear almost identical in other respects.

Generally, if blasting has been anticipated, the Contractor will be required to retain the services of a qualified blasting control consulting engineer to review and approve the proposed blasting method, recommend precautionary measures, and submit a report on these matters to the owner or his representative. This may include initial test blast monitoring and recommendations resulting therefrom.

The specifications may also define the degree and frequency of further monitoring to be provided, from continuous full-time service to intermittent service at specified intervals of time of distance, or at any change in pattern, method, or explosive quantity.

A pre-blast survey of adjacent vulnerable properties may be specified in the contract documents but even if it is not, the contractor may have such an examination undertaken on his behalf, either at the instigation of his Insurers or on his own initiative.

Most specifications will stipulate a maximum permissible ground vibration intensity level at the nearest vulnerable structure or service. If the Model Municipal Noise Control By-Law is not applicable or specified, the restriction mentioned will most likely be the peak particle velocity limit of 50 mm/sec given as a guideline in O.P.S.S. 515.

It should be noted that the 50 millimetre limit specified in O.P.S.S. 515 applies to completed buildings and services. The limit permitted next to concrete or grout in place less than 60 hours is 10 millimetres per second peak particle velocity.

BLASTING TECHNIQUE

EXPLOSIVE TYPES

Commercial explosives used today are of four main types, differing in chemical composition and in the physical properties they exhibit as explosives. The four types are:

1. ANFO - Ammonium Nitrate/Fuel Oil dry blasting agent is the most widely explosive in the world today due to its low cost. However, it has not seen much in sewer trenching since it dissolves and completely desensitizes in water.

2. Dynamite - An explosive containing nitroglycerine as a fuel/sensitizer is generally referred to as a dynamite. Dynamites have been the mainstay of most trench blasting since they possess good water resistance and are extremely reliable. They are packaged in paper shells.

3. Water Gel (Slurry) - This types uses an alternative sensitizer to nitroglycerine. Water gels are less sensitive than dynamites and, because of this, were heralded as the new "safer breed of explosive" when they first became commercially available in the early 1970's. Water gels are packaged in plastic film cartridges and have good water resistance.

4. Emulsion - This is the latest type of commercial explosive and differs from the others in that it exhibits a very high velocity of detonation. This characteristic makes emulsion a very efficient explosive product. Emulsions are packages in plastic film but will also be available in paper shells.

Initiation Systems

An explosive requires an external source of energy in order to make it detonate. In blasting operations this is routinely accomplished by means of commercial detonators (ie. blasting caps). The detonator is actuated at the appropriate moment by the blaster and provides sufficient energy in the form of sudden impact to detonate the explosive.

Initiation systems (ie. detonators) used in sewer trenching are of two types: electric and non-electric.

1. Electric detonators (short delay) are still one of the most commonly used initiation systems for open cuts (trenching). Originally the time interval separating successive delay periods ranged from 25 ms (milliseconds) in the lower period to 75 ms in the higher periods, but recently a more consistent and exact series of delay periods with equal time lapses has been developed. Delay detonators allow for the systematic relief of the trench rock being blasted. This produces better fragmentation, better control, less flyrock, less noise, and lower vibration levels than a blast where all holes are fired instantaneously.

2. Non-electric detonators, known commercially as NONEL Detonators, are growing in popularity since the development of the "shock-tube" method of initiation. Originally they were used less often than electrical caps, generally when an electrical hazard existed as a result of the presence of nearby hydro lines, radio transmitters, etc., where stray current or radio signals might infiltrate an electrical circuit and cause a premature detonation. NONEL was not widely used in built-up areas because detonating cord was required to initiate the detonators, and this created appreciably more noise than electrical initiation. Recently developed NONEL systems however, involving surface delays of 25 millisecond periods and down-hole delays of varying durations, are easier to hook up and have added advantages including the facility to detonate many more holes per blast while maintaining a limit on the number of holes per delay period.

While short delay detonators are used in open cuts, tunnel blasting is generally done with long delay electrical detonators. The time interval separating successive delay periods ranges from 300 ms for the lower periods to 700 ms for the higher periods. Long delay detonators are used in tunnel headings because of the very confined nature of the blast. Relief holes are drilled in the "cut area" and left unloaded to provide a void for the initial holes to break to. Allowing longer time periods between the firing of subsequent holes gives sufficient time for the blasted rock to be thrown clear of the blast, thus preventing the broken rock from "freezing". The result of using long delay detonators in tunnel blasting is better fragmentation, longer advance per round, less overbreak on the walls and back, and lower blasting vibrations.

BLASTING SAFETY

Many of the rules for maintaining safe practice on a blasting site are based on simple common sense. The reasons for the "no smoking near explosive products" rule for instance must be readily apparent. In fact, nitroglycerine (the sensitizer in dynamite) detonates at about 220 degrees Centigrade. The tip of a cigarette burns at 800-1000 degrees Centigrade. Similarly, the necessity for maintaining safe distance in drilling near loaded holes is obvious. The pace of a drill bit on an undetonated remnant of explosive in a "bootleg" has probably resulted in more fatalities than all other blasting-related fatalities combined.

Wherever blasting operations are undertaken, the safety of all personnel within the blast area must be the primary consideration in carrying out the work. It is useful to discuss safe blasting practices under three headings:

1. Safety Before the Blast - Transporting of Explosives - The safe transportation of explosives from a manufacturing plant or main magazine depot must be carried out in strict accordance with established regulations. The TRANSPORTATION OF DANGEROUS GOODS ACT (T.D.G. Act), which became federal law in 1985, regulates the transportation of all hazardous materials, including explosives, whether by road, rail, air or water. Wherever the T.D.G. Act Regulations are silent, the federal EXPLOSIVES ACT Regulations apply with respect to transportation.

The contractor who transports explosives from a nearby magazine to his job site by public road must comply with the regulations pertaining to both of these federal acts.

Most accidents that have occurred during the transportation of explosives to the job site have been the result of unsatisfactory loading of the vehicle, collision, or fire. While such accidents have been remarkably few, most have been the result of carelessness or a flagrant disregard for regulations.

The regulations for the licensing of a truck to transport explosives, etc. are outlined in the EXPLOSIVES ACT Regulations. In addition, it should be noted that explosives and detonators must be transported separately unless proper precautions have been taken to isolate the detonators from the explosives by means of an approved barrier.

If the contractor sets up his own magazine near the job site he must apply for a Magazine License to the Chief Inspector of Explosives, Department of Energy, Mines and Resources, Ottawa. The license must be posted inside the magazine where it is clearly visible. A log book must be kept to record shipments received from the explosives supplier, quantities taken out and transported to the job site, and quantities returned unused from the job site at the end of the work day.

Explosives stored in a magazine must be neatly stacked according to size and grade, not above the "stack line" nor to block floor or wall ventilators. A corn broom should be kept in the magazine for the purpose of sweeping it out at regular intervals.

Explosives and detonators must be stored in separate magazines. If primacord is used in blasting operations, it must be kept in the explosives magazine, not the detonator magazine.

An adequate supply of proper stemming material should be on hand, as should an adequate supply of blasting mats if the lack of a sufficient depth of overburden requires that these be used. In order to properly place blasting mats, the blaster will have to arrange for a suitable piece of equipment to place them over the blast without disturbing the blasting circuit(s). A backhoe is commonly used.

2. Safety During the Blast - The number of people on a blast site when the actual loading is being done should be kept to a minimum and in most cases will consist solely of those doing the loading. As far as determining what constitutes the "blast site", no hard and fast rule exists. In general, it must be considered as that area in which flyrock from the blast could cause injury to personnel or damage to equipment when the blast is fired. For practical reasons during loading, it may simply be the immediate section of trench that has been drilled and is being loaded with explosives.

As soon as explosives are moved onto the blast site, the area should be marked off with danger signs to keep unauthorized persons away. If electric detonators are to be used, signs requesting motorists to turn off their radio transmitters should also be posted.

Loading and wiring-up explosives in a thunderstorm is particularly hazardous and must be avoided. The hazard of premature detonation by lightning strike is appreciably higher in electric blasting, but non-electric detonators may also be initiated by a direct strike. Commercial "lighting detectors", which indicate the approach of a thunderstorm and its distance from the site, are now available and their use is now widely specified for certain blasting activities, eg. on large-diameter natural gas pipeline construction projects.

When ready to fire the blast a simple check list should be established and followed religiously:

- determine the "blast area" and make sure all personnel and equipment are removed.
- select guards to patrol all paths of entry to the blast site.

- provide cover for any personnel or equipment that must remain on the blast site during the blast.
- provide a warning signal audible at a distance of one kilometre from the site in compliance with the current requirement of the Occupational Health and Safety Act and Regulations for Construction Projects.
- the blaster should be the last one to leave the blast site and should fire the blast only when he is absolutely certain that it is safe to do so.

Also worth noting is the fact that local residents view blasting operations with some trepidation. A well established blasting procedure and adequate warning of an impending blast will do a lot to alleviate their concerns.

3. Safety After the Blast - Immediately after firing an electrical blast, the blaster must disconnect the blasting leads from the blasting machine terminals and shunt the leads out by twisting the ends together. This is a safety precaution to ensure that no current can be accidentally applied to the lead wires when the next blast is being hooked up. In addition, an "all clear" signal is sounded by the blaster to indicate that it is safe to let local traffic proceed.

It is the responsibility of the blaster to thoroughly inspect the blast site for any signs of misfires or any other unsafe conditions. If a misfire is found, the correct way to remove the hazard is to reprime and reblast the undetonated explosive. First, however, a reasonable length of time should be allowed to elapse before proceeding - one hour is considered adequate. Any stemming or non-explosive debris should be removed by hand or with the aid of a non-sparking tool (ie. wood or brass scraper). Once the undetonated explosive has been exposed, a fresh primer cartridge should be placed on top and the charge stemmed and matted as would normally be done to control flyrock. The mishole should be fired right away.

Sometimes a cloud of orange/brown fumes is produced by the blast. This is usually associated with a blast loaded with ANFO where water was present. These fumes are toxic and contact with them must be avoided until they disperse in the atmosphere.

The last row of holes blasted should be referenced by the driller prior to blasting so that he will know where to begin drilling the next blast. In a trenching operation where a deep rock cut necessitates blasting in two lifts, great care must be taken by the driller to make certain that he does not drill into a "bootleg" when he sets up to drill on the lower lift. This should be done by removing all debris from the rock surface with compressed air and washing it down with a hose.

It has been demonstrated on countless jobs involving many millions of tons of rock blasting that accidents can be avoided by following the established safety rules, not taking short cuts, and treating the explosive materials with the respect they deserve.

SUBJECT:

CONSTRUCTION OF SEWERS
WATERMAINS AND ASSOCIATED
FACILITIES

TOPIC: 13

TUNNELLING

OBJECTIVES:

The trainee will be able to:

1. Recall the reasons for tunnelling.

2. Identify and understand the different methods of tunnelling and the problems confronting the inspector when various methods are employed.

3. Understand that conditions vary widely across the Province and realize that the inspector should first concentrate on conditions in his own area.

TUNNELLING

INTRODUCTION

Tunnelling is an expensive way of providing an underground service and there are two main reasons for using this method, namely the size of depth of the service. The open-cut installation of a pipe of 2400mm diameter or larger in a built-up area would make it impossible to maintain services of water, sewers, hydro, gas, telephone, fire, ambulance and police, and would completely disrupt the area. The depth required may be greater than can be tackled by open-cut.

Glossary

1. **Primary Lining:** Support system for ground prior to the installation of the secondary or final lining. In use for a short time in hand mining. May be in place for several weeks in machine mining.

2. **Liner Plate:** Circular steel liners assembled in sections and bolted together. Plain black liner is used as temporary lining. Galvanized heavy plate may be used as permanent liner.

3. **Grouting:** The application of cement - sand mixture under pressure to fill voids behind primary or permanent lining.

4. **Spring Line:** Horizontal Centre line of the tunnel.

METHODS OF TUNNELLING

1. Hand mining
2. Machine mining
3. Pipe jacking
4. Augering

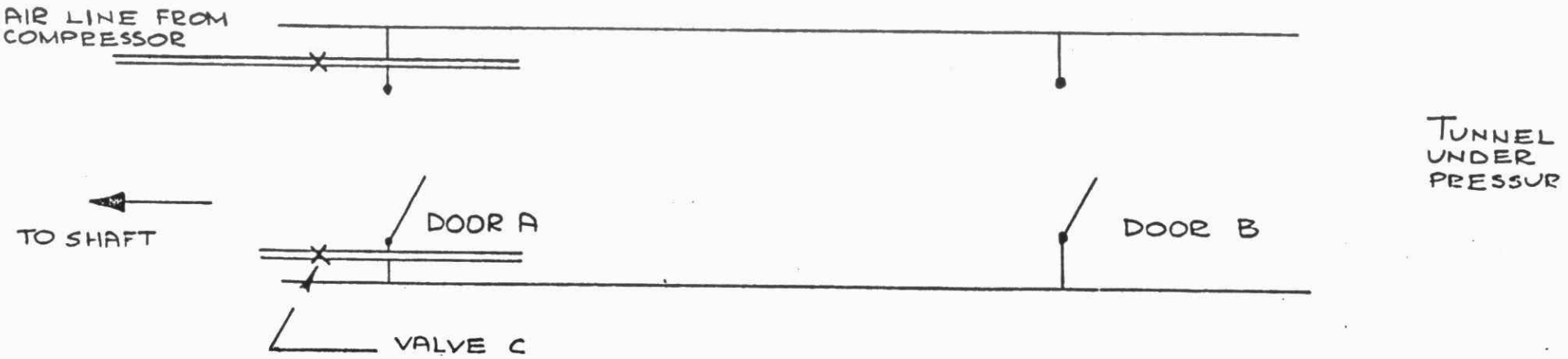
Shafts

All of these methods require a shaft for access to the work. A sound, safe shaft is of prime importance in all tunnel jobs, however big or small. In soft ground shallow shafts are often supported by timber walers and sheeting. It is very important to see that no ground is lost behind the sheeting. A good concrete collar around the top of the shaft to keep water out is a help. In soft or wet ground a circular shaft lined with liner-plate has the advantage of being bolted tightly together and grout can be placed behind it. This should be done each day during sinking of the shaft. If the shaft is deep or the ground is doubtful, it is an advantage to have the shaft at ground level larger than is required for the manhole or structure to be erected. This allows the use of another method if conditions deteriorate. Sheet piling or H-Beams and lagging may be sufficient to enable the shaft to be completed. If the water problem is serious, the water table must be lowered. Well points or deep wells may solve the problem if the ground conditions are right. As a last resort a vertical compressed air lock may be used. Mains or services near a shaft may cause problems the whole time the shaft is in use. If there is a sewer or watermain close to the shaft, it should be checked and, if necessary, protected so it will not be damaged in the course of the work. If, when exposed, it is found to be defective, a local municipality or public utility will have the responsibility to repair it. If it is not exposed and is later found to be damaged, it will be difficult for the contractor to escape responsibility. When the bottom of the shaft is reached, a good working slab should be poured if the shaft is to be used as a mining shaft. This should be kept below the structure to be built in the shaft.

Hand Mining

In soft ground it is normal to mine one or two shifts and place concrete on the second or third shift. The primary lining has only to support the ground until the concrete grains sufficient strength to take the load. In good ground steel ribs and wooden lagging are used. In this case, it is often possible to mine ahead for the full length of the plank before it is installed. When conditions become worse, poling plates that are jacked ahead as the excavation advances can be used. These plates must be installed in advance of the bad ground and carried along until required. Precautions should be taken to avoid the plates sticking to the concrete and being lost to the job. The plates should be oiled and may need to be moved ahead slightly each day when mining is not taking place. If a curve is to be negotiated, the radius of the curve has to be taken into consideration in deciding the maximum length of plate that can be used. These plates support the top half of the tunnel. If the invert is soft or running, planking may be used in the invert or complete rings of liner plates may be required. These need to be grouted on a regular basis.

A last resort is the use of compressed air to hold back water, approximately 10kPa of air pressure per metre head of water. The diagram of the airlock shown (Figure 13-1) is simplified to show only the principles involved. Prior to entering the lock, pressure in the tunnel keeps Door B closed. The lock is entered, then Door A is closed and air is pumped into the lock. The increase in pressure holds Door A closed. When the pressure equalizes, Door B will open to allow access to the tunnel. On return Door B is closed after entering the lock and the air is exhausted from the lock to the outside through Valve C. Door A will then re-open to allow return to the shaft. This method is not used except as a last resort since high medical standards restrict the number of men available to work in compressed air, and reduced working hours as the pressure increases raises the cost considerably.



PLAN OF AIR LOCK

Figure 13-1

In hand mining the excavation will not exactly conform to the primary lining therefore there will be voids to be filled by pressure grouting. Grouting is not considered a high priority by most contractors because they tend to stress progress as so many feet per day, and this might be slightly reduced by the time taken to grout. The frequency with which grouting is required depends on the type of ground. Agree on a regular schedule of grouting with the Contractor. Watch carefully for voids and lost ground, as if not caught quickly they will eventually show up on the surface. It will funnel quickly to the surface in sandy ground.

Ground conditions can change daily so the inspector should keep a close watch on the situation and discuss any changes with the Contractor.

A bulkhead is required at the face at the end of each shift. The strength of the bulkhead will depend on the stability of the ground and the length of time that it is going to be left in place. The bulkhead should be checked regularly when the job is closed down for any reason.

Mining Machines and Shields

A shield is a steel tube that is pushed ahead of the primary lining by a series of hydraulic jacks and provides protection for the work. The primary lining is built inside the tail of the shield and may be expanded to reduce the gap between the lining and the ground after the shield has moved ahead. If cast iron or concrete segments are built behind the shield as a permanent liner, they must be grouted. By closing the face, the ground is prevented from running in. Mining is done by hand. This gives a better and safer job but not necessarily a quicker one. Shields can be used under compressed air. Grade checks are important because changes cannot be made quickly.

A full-face mining machine cuts the ground with revolving cutters and deposits the muck onto a conveyor. The machine is moved in the same way as a shield. The primary lining is erected in the tail of the machine. It is often ribbed with wood or concrete lagging. The efficiency of a machines operation is governed by the speed with which the muck can be moved from the back of the machine. In most cases, the machine is able to dig far more muck than the equipment behind it can move away. Well-placed switches, big trains, powerful locos and muck storage areas near the top of the working shaft are required for maximum efficiency. It is preferred to leave the primary lining alone as long as possible to allow maximum working room. In any mining operation, rocks in the face are unpredictable. Test boreholes are too far apart to give a fair indication whether or not boulders will be encountered. If the rock is too large to come through the face of the machine, it is a slow and tedious job to break it up. A rock in the invert is more difficult to deal with than one at spring line or above.

An Alpine Miner is a mining machine that is independent of the shield. It has a single arm with a cutting head which can be used to cut a circular or square section. When used with a shield, a control ring can be used to enable an exact circle to be executed. Any mining machine operates best in homogeneous ground as a mixed face can cause considerable problems because the cutting device which is good for one part of the face may not operate as well on the other part. A mixed face of clay and rock is a very difficult face for any machine to tackle.

Pipe Jacking

In principle, pipe jacking can be compared to mining with an open-face shield. The front pipe has a cutting ring mounted on it acting as the shield. In the shaft there is a jacking block sufficient to take the strain of pushing the number of pipes necessary to complete the heading. The hydraulic jacks push one pipe length, are then withdrawn and a further pipe inserted. Bentonite grout is pumped through a hole in each pipe to make it easier to jack. On days when the jacking and mining operation is not taking place, the jack should be operated to move the pipe a few cm. This will avoid start-up difficulties after a delay due to breakdown of the equipment or a holiday. Because of the movement of the pipe the survey checks have to be made from the shaft by laser or instrument. Accuracy is essential since to correct line or grade is very difficult. If water is encountered, deep wells or well-points may be required. The difficulties of working through a vertical airlock make the use of compressed air almost impossible.

Augering

This operation is normally used for small, shallow pipes of 300 to 375mm diameter under roads and rights-of-way.

Twelve metres is the limit with a reasonable chance of maintaining line and grade. Local mains and services are sometimes a problem. A small percentage of holes may need to be abandoned and re-drilled. Since the hole is augered and the pipe placed without workmen entering the excavation, the type of pipe joint is important.

The joint has to be made and the pipe then moved into the hole. The length of pipe and therefore the number of joints, is governed by the maximum size of shaft possible. In soft or sandy ground a casing is needed and the pipe is grouted in. The casing is jacked in with the auger and is kept just behind it to prevent loss of ground. The latest developments in this field allow for boring of pipes up to 900mm diameter.

Rock Tunnels

A tunnel excavated by blasting is similar in principle to hand-mining. Removing loose rock by scaling is important. If the concrete is not poured close to the face, rock bolts will need to be installed in a pattern suitable to the type of rock. Steel wire mesh may be used as an additional safety precaution. The inspector should know and check the blasting pattern, method of loading and size of charge to be used. It is important in a built-up area that the amount of explosives used be controlled to reduce the possibility of damage to private property. Seismic tests are usually made on a regular basis and the inspector should be familiar with the appropriate clauses in the Specification and ensure that they are followed.

PUBLIC RELATIONS

As the tunnelling shaft may be outside a taxpayer's property for as long as two years, care should be taken to provide an atmosphere in which people can live. The inspector is the key man because he is the Owner's representative who is seen all the time. It is helpful if politicians are briefed ahead of time. An Alderman who understands the problems can be of great assistance. The inspector is responsible to see that the working area is set-up as required by the Specifications.

At the time of start-up it is useful to have some improvements made that can be left. Check any changes that may be required because of local factors. Some people can never be satisfied. Never improve the situation for a complainer at the expense of a reasonable taxpayer. Encourage people to explain their problems on site before calling their Alderman. A project involving explosives is a special problem. An inspector by insurance representatives is usually made prior to the start of the job. A letter explaining the project to the local inhabitants should be sent out by the Owner, municipality or government department. A quick response to complaints should be made. Do not get involved in any claims; always ask that these be made in writing.

INSPECTOR'S DUTIES

These are not difficult as at first they may appear. One of the most difficult things to get used to is not being able to see where you are going. Some inspectors turn down a tunnel job before studying carefully what is involved.

An inspector's first experience in tunnel work is often obtained on a project that consists mainly of open cut work but does have some small tunnels under roads and rights-of-way incorporated in it.

It is easier for a man with mining experience to become a tunnel inspector. However, it is possible for a concrete inspector to work in a tunnel with the concrete shift and then enlarge his experience to include mining.

Hand Mining

Check line and grade and size of excavation to ensure that it is large enough. Safety of the primary lining is important. Check support of the lining, particularly the centre rib if two planks are used and the foot of the centre rib if a full rib is not used. Watch for changes in the type of ground or for wet conditions as more or stronger primary lining may be required. On the concrete shift, check the invert when forms are stripped, as it quickly becomes covered with dirt on the next mining shift. Check that forms are clean and oiled before re-use. Check line, grade and clearance.

In bad ground make sure you know the tolerance allowed in concrete thickness before reinforcing steel or later remedial work is required. Sometimes, the pour is allowed because conditions are becoming worse. Further mining should not be allowed until a method to overcome the problem is agreed upon.

Curing conditions are normally good in a tunnel. Encourage patching and grouting as the job proceeds.

Machine Mining

Check line and grade which is normally set by laser. Check the type of ground being excavated. It the machine has gates that can be used to close off the face, check that these are closed when the machine is not operating. When the concrete is kept back from the machine, check for any sign of strain in the primary lining. The concrete operation is basically the same as other methods except that the pours are much larger.

Pipe Jacking

Check line and grade if set by laser or ensure that the surveyor makes regular checks from the shaft. Check that there are no signs of damage to intermediate pipes or joints. Check for type of ground and any loss of same.

Safety

Safety is a bigger problem underground. We are dealing with heavy loads and equipment in a confined space, and great care is required. We rely on artificial light and must be prepared for a power failure and use of emergency lighting. We must watch for moving trains. Hands and feet are very vulnerable where concrete forms, conveyors and muck cars are concerned.

The Occupational Health and Safety Act is enforced by Provincial Safety Inspectors who visit projects approximately once a week. This does not relieve the resident inspector of a moral responsibility for a safe operation. The interpretation of the Act can be discussed with the Provincial Inspector on his visits to the site.

Surveying

When the tunnel is on a straight line and steady grade, checking is comparatively simple. In small tunnels the line is set from vertical strings hung from the crown, and grade is checked by horizontal strings looped on to spads at the sides of the tunnel. The inspector must remind the surveyor to extend line and grade closer to the face before sighting becomes difficult.

When a job is proceeding will and steady progress is being made the surveyor will have a regular schedule for his work.

Survey on a curve is more difficult. It is of the utmost importance that you understand the method that is being used to lay out the curve. There are several ways that the line can be set ahead on chords or tangents to the curve. To check properly it must be carefully explained by the Engineer or Surveyor.

When a laser beam is being used it must be checked regularly to ensure that it has not been knocked or moved by vibration. The surveyor will provide the information necessary to check laser alignment.

SUBJECT:

CONSTRUCTION OF SEWERS WATERMAINS AND ASSOCIATED FACILITIES

TOPIC: 14

SANITARY AND STORM SEWERS

OBJECTIVES:

The trainee will:

1. Be aware of the purpose of:

   a) A sanitary Sewer System
   b) A Storm Sewer System

2. Discuss as related to flow in sewers and to sewer construction:

   a) Grade and Alignment
   b) Manholes
   c) Complex Sewer Structures
   d) House Service Connections
   e) Catch Basins

3. Recall important points that should be considered during the inspection of a sewer construction project.

SANITARY AND STORM SEWERS

GENERAL ASPECTS

The purpose of a sanitary sewer system is to collect flow from all points of a developed catchment area and transport it to a treatment facility. The design flow for a sewer is calculated from the population density or number of people per hectare in the area served. The design flow calculated per hectare will vary depending on the use of the land, i.e. residential, commercial, industrial.

The purpose of a storm sewer system is to remove and direct stormwater to a suitable outlet, usually a creek, river or lake. The design flow for a storm sewer is calculated from the run-off produced in the catchment area served by the sewer from a storm of a pre-determined intensity. The design flow will vary in quantity depending on the nature of the ground cover in the catchment area. Parking lots give almost instantaneous run-off, whereas grassy areas will absorb much of the rainfall.

The hydraulic elements of sewers include the determination of the sizes, depths, shapes and materials of the proposed construction.

GRADE AND ALIGNMENT

Sewers must be laid to line and grade and remain on line and grade to ensure efficient operation and avoid unnecessary maintenance and general operating problems. This can be accomplished by using grade stakes to which batter boards and string lines are set. Modern sewer construction involves the use of lasers which are set to the specified grade of the pipe.

Local service sanitary sewers are usually located in the centre of the street permitting a symmetrical arrangement for the private drain connections from the houses serviced by the sewer. These sewers must be deep enough to allow each connection to be laid at a proper grade and are themselves laid at a grade which will ensure that the sewage flows at a specified minimum velocity in times of low flow. The minimum velocity is that calculated to ensure transportation of all matter in the sewage at all times.

Local storm sewers are generally larger than the sanitary and need not be laid at such a depth as they generally collect the flow from catch basins. Their size depends on the size of storm setting the design criteria.

In both cases, the local sewers carry their flows into larger collector sewers which, in turn, pass the flow into interceptor sewers and thence to the treatment facility or point of discharge. These larger sewers are still laid in open cut but may be driven in tunnel if large enough or if dictated by economics or other considerations.

Contractors will generally use a transit to keep the pipes on line and grade. Often a laser beam is used which is set in the downstream manhole and projects through the pipe to a target placed in the bell of the pipe being set. This provides for very rapid progress of the work. Some contractors who have used this method claim to have achieved at least twice their daily production. Lasers are very common in tunnel work, especially where a mining machine is being used.

Bedding the pipe and backfilling the trench are of equal importance to line and grade. The bedding must be placed on firm undisturbed ground and compacted before the pipe is placed on it to ensure that the pipe does not settle after backfilling. The first stage of the backfilling procedure is the placing and compacting of suitable material around and at least 300mm above the pipe. This is required to ensure that the walls of the pipe are supported to the sides of the trench. When a pipe is laid in such a well supported manner it will be structurally capable of carrying the remaining backfill dead loads and surface live loads. It will not be displaced during backfilling, it will be stable in the event of excavation adjacent to it at a later date. When completed, a good job will look like a gun barrel.

MANHOLES

One of the most important and most common structures found in both storm and sanitary sewers is the manhole. Manholes are placed at specified locations for cleaning and inspection purposes, at all changes in grade, changes in direction, changes in pipe size or variation in quantity of flow.

It is common practice to have a manhole at the upstream end of a sewer for convenience in flushing and cleaning and in the event of a future extension of the sewer. The recommended distances between manholes on a straight run is as follows:

1. 90m for sewers 375mm diameter or less
2. 120m for sewers 450 to 750mm diameter
3. for sewers greater than 750mm diameter the spacing may be increased especially where sewers are big enough where a man can actually walk through. The spacing, however, also depends on the cleaning equipment which the municipality has. If modern and efficient cleaning equipment is available in the municipality then the manhole spacing may be greater than a similar municipality which might have poor cleaning equipment.

Manhole spacing is governed by grades available. If flat grades are necessary then manholes should not be spaced too far apart since blockages are more likely than if good sewer grades are possible. At the other extreme, sewers going down steep hills may require "drop" manholes, closely spaced, to dissipate energy.

Manholes can be poured-in-place or precast and be designed as a drop manhole if a drop in excess of 1m occurs between any invert and the lowest invert in the manhole. The size of the manhole is usually determined by the size of the largest pipe entering the manhole but seldom are poured-in-place manholes less than 1200mm square and precast manholes less than 1200mm in diameter. Deep manholes will have safety gratings placed at suitable vertical distances, maximum 4.5m. These gratings are hinged to ensure that they stay in place but can be lifted to allow removal of foreign objects and to facilitate access to the sewer.

Many municipalities have standard details for manholes, the types of sealant acceptable in precast manholes and recommended locations for items such as step-irons. Aluminum ladders are preferable to step-irons but the first and last rungs must be located near to the cover at the surface and the benching at the bottom and the ladder located in the manhole in a position that will not interfere with sewage flow. Benching is an important part of the manhole as lack of it or incorrectly shaped benching causes head losses and sedimentation. Benching should also be sloped to ensure cleaning of deposits from high flow occurrences.

Excavations for manholes must be properly supported to maintain stability of the surrounding ground. This will prevent possible displacement of the manhole after backfilling, especially in the case of deep manholes. The manhole must sit on a solid foundation to prevent settlement of any kind. The pipes entering the manhole will have joints placed close to the outside face of the manhole to provide some flexibility and allow for differential movement between the pipe and the manhole. Backfilling should be done in lifts to permit good compaction as only too often road surfaces settle around the manhole cover as traffic compacts the backfilled material. All manholes should have corbelling or adjustment rings to permit variation of elevation of the manhole cover if required due to street re-design. The drop pipe in a drop manhole should be surrounded with concrete poured with the manhole walls to ensure a homogenous construction.

COMPLEX SEWER STRUCTURES

Complex structures are included in certain sewer systems. Vortex shafts can be used to transfer sanitary sewage from collector sewers to an interceptor sewer. Drop shafts can be used for the same purpose in a storm sewer system. Such structures are designed with energy dissipating features. A vortex chamber has a stilling basin ahead of the drop shaft as the flow must enter the shaft in a specific manner. A drop shaft will have a submerged outlet or a break platform or a dissipation block.

Storm sewer systems have outfalls to lakes and rivers. Often these are underwater as such a design prevents problems with freezing or bank erosion. Outfall structures above water level must have safety gratings to prevent illegal entry, gabions or some form of bank protection, some form of energy dissipation feature, and a pleasing appearance to the surrounding neighbourhood.

HOUSE SERVICE CONNECTIONS

The general purpose of constructing a sewer is to allow connections to be made to it from abutting properties. The flow characteristics in the sewer can be affected by the manner in which connections are allowed to be made and the number and relative size of connections made. Wherever possible, factory-made tee junctions should be installed at the time of original construction and the designer should made every effort to anticipate the number of connections required. In excessively deep sewers, it is quite important that risers be installed at below finished grade. Nowadays storm and sanitary systems are separate and it is imperative that a method be devised to make sure that private drain contractors know the difference between the storm and sanitary connection lines and are made to realize the importance of connecting properly. In spite of regulations and inspection, there is an alarming amount of storm water entering into the sanitary sewers. No amount of careful design can affect poor control over use of connections. On well-inspected sewer construction, sewers are adequately tight against infiltration, yet each new sanitary sewer, after private connections are made, has a marked increase in flow rates during and immediately after a rain storm. close supervision of construction of house service connections is necessary and desirable.

CATCH BASINS

One of the important units in a storm sewer system is the catch basin. Regardless of the adequacy of the underground sewer system, proper drainage cannot result unless storm water is quickly and efficiently collected and passed into that system. The purpose of the catch basin is to drain away surplus water from a road allowance or parking lot. Generally, catch basins are installed at intervals of 75 to 90m on both sides of a street.

Until recently, it was customary for a designer to show that standard catch basins were to be installed at various points. Today, there are several types of catch basins. There variations developed from observing problems in the field and the designer should consider the conditions likely to prevail and choose the type of catch basin that will best suit the given situation.

The most widely used storm sewer inlets are as follows:

a) Gutter inlet - a standard catch basin sitting in a slight depression in the roadway gutter.

b) Arch-back inlet - a catch basin inlet composed of both curb and gutter openings acting as an integrated unit.

c) Ditch inlet - an oversized catch basin with its frame usually inclined at 30-45°.

Catch basins with flow restricting devices can be purchased for use in areas requiring stormwater management controls.

Catch basins are usually constructed with a 300 to 450mm sump in the bottom which collects grit, leaves and other debris which otherwise would enter the sewer system. Catch basins should be cleaned regularly to prevent sewer blockages.

MATERIALS USED IN SEWER CONSTRUCTION

The following factors must be considered prior to selection of the sewer construction materials:

1. Flow characteristics
2. Life expectancy
3. Resistance to scour or sulphide build-up
4. Resistance to acids, alkalis and solvents
5. Ease of handling and installation
6. Structural strength
7. Type of joint
8. Availability of pipe and size variations

The four most commonly used materials are plain or reinforced concrete pipe, poly-vinyl chloride pipe, polyethylene pipe and vitrified clay pipe.

CONSTRUCTION CONSIDERATIONS

There are certain important points that an inspector should pay particular attention to during construction. Some of these are as follows:

1. The bedding must be to grade and a hole dug out for the bell to prevent bridging of the pipe from bell to bell on the bedding.

2. Final adjustments may be made to the pipe grade by placing pieces of wood under the bell or front edge of spigot piece but these must be removed when grade is set.

3. Ensure that materials are properly compacted around and over the pipe.

4. There should not be large boulders in the backfill.

5. Place concrete under inlet/outlet pipes between base of manhole and natural excavated ground or install a flexible joint within 300mm of the base of the manhole.

6. Ensure that there is proper connection of house services at the main sewer and a proper plug in the end of the service, especially where connecting to an older sewer. Careless or improper connections might block the passage of T.V. cameras, making full inspection impossible.

7. A very important duty of the inspector is to ensure that the joints are made properly whether they are confined 'O' ring or collar type in that the bell and spigots are in good shape, not damaged, are perfectly round and that the pipe is seated properly. But fused joints of polyethylene pipe should be checked for process and uniformity.

8. Smaller pipes are usually put home by hand with a large bar. They must be fully driven home and not permitted to spring back. Larger pipes should be put home with a come-along operated from inside the pipe. In no case should the backhoe bucket be used to drive a pipe home.

SUBJECT:

CONSTRUCTION OF SEWERS
WATERMAINS AND ASSOCIATED
FACILITIES

TOPIC: 15

WATERMAINS

OBJECTIVES:

The trainee will be able to:

1. Recall the purpose of a watermain system.
2. Discuss in general terms the distribution system layout and alignment.
3. List watermain appurtenances and recall the purpose of each.
4. Recall important points that should be considered during the inspection of a watermain construction project.

WATERMAINS

GENERAL ASPECTS

A watermain system is designed to adequately satisfy the water requirements for some combination of domestic, commercial and industrial users as well as fire-fighting purposes. The system should be capable of satisfying the maximum demands placed upon it.

A watermain system is composed of three main parts:

1. Source plus treatment - lake, river or well plus necessary purification facility.

2. Transmission line (feedermain) from facility to distribution system.

3. Storage - in a ground reservoirs, elevated tanks or stand pipes.

4. Distribution system to customers.

SYSTEM LAYOUT AND ALIGNMENT

A water distribution system is generally divided into pressure zones. Each zone contains a network of pipes capable of transmitting water flows at the specified zone pressure. Generally, each zone will have a pumping station to overcome the line losses and maintain pressure, and a reservoir to balance the flows. The distribution system should have certain design features to reduce losses within it:

1. Minimize dead-ends by looping mains, or provide flushing hydrants.
2. Interconnect the various pipelines forming the system at regular intervals - 350m or less.
3. Locate valves to minimize the area that would be deprived of water service in the event of a shut-down.
4. Space hydrants in accordance with the Canadian Underwriters' Association or the local fire department or at no more than 100m intervals for local streets.

Many municipalities have standard street cross sections which detail the location of a watermain in any one street. Generally, the watermain is located off the travelled portion of the roadway, in the boulevard, as this reduces installation costs and provides more insulation for the main as frost penetrates deeper beneath a travelled roadway. In the event that the street is a major thoroughfare or highway, the main will be located will off the pavement or even in adjacent property or streets as traffic disruption and road restoration are major considerations and better avoided.

The depth of cover can generally be kept to a minimum, for a watermain being a pressure conduit. Considerations required in this part of the design are the depth of frost penetration, the use of the main (continuous or otherwise), trench loading and proposed or future road grades. Generally, the minimum cover is 1.5 m. In instances where this minimum cover of 1.5m cannot be attained the pipe line could be insulated. To determine the insulation; several factors including degree of frost penetration, depth of cover, type of soil, flow in the pipe and ambient temperature.

In the case of all watermains, a plan has to be prepared which details the location of all utilities. This information permits a study of the street to determine whether or not there is space available for a new installation. Once it has been decided that such a route is feasible, drawings are prepared and submitted to all interested authorities or utilities for their information and general approval.

It is important that the watermain be laid to the horizontal and vertical alignments chosen in the design. As a pressure conduit a watermain has be give way to storm and sanitary sewers, and their connections, and to major installations of other utilities such as underground hydro vaults or gas metering chambers which are also generally located off highways. Construction consideration has to be given to overhead wires and trees which are often close to or over the proposed route. Poor attention to alignment may cause air pockets to form in the main which cuts down the effective pipeline area. The inspector will notice that any watermain, large or small, has often many tight squeezes along its route caused by these existing conditions.

One of the benefits that a watermain possesses, being under pressure, is its inherent flexibility with regard to its alignment. Vertical shafts and tunnels can be incorporated to traverse valleys or steep slopes, roads and railways can be tunnelled, rivers can be crossed in open-cut or on piers, swamps or muskeg can be overcome by rafting or piling. In many cases a watermain project can be a challenge to the inspector.

WATERMAIN APPURTENANCES

A trunk watermain or watermain system requires various aids to allow efficient operation or control, generally termed valves and appurtenances. These are discussed below.

Control Valves

These are spaced to minimize the area that would be deprived of water service in the event of a line break or a planned or emergency shut-down.

Gate valves are generally used in distribution mains. It is common practice to install four valves at a four-way intersection and three valves at a three-way intersection. Valves would be spaced at a maximum distance of 150m at other occasions but this will vary in accordance with the density of land use.

Butterfly valves are often used in trunk mains as they cause less pressure disturbance in opening or closing. These would be placed at zone changes, municipal boundaries or between major distribution main junctions. These are also used in distribution mains larger than 300mm diameter.

The specifications should give the direction of valve opening which should be consistent with the bulk of valves presently in the system or as approved by the client. The size of operating nut or type of operator should also meet these requirements.

When a connection is required to an existing system, often a tapping sleeve will be used. A pressure tap consists of a tapping sleeve which is braced around the pipe to be cut into and a gate valve which is attached to the sleeve to prevent the water from the pressurized pipe escaping after the connecting hole is cut in the existing pipe. Pressure taps are most common for small diameter connections and not used for very large connections as the tee must be reinforced if the circular structure of the pipe is excessively reduced by a large diameter tee connection.

Air Relief Chambers

A watermain can carry considerable amounts of air and water. This is not a serious problem in a system with many fire hydrants as these can be used to relieve air build-up in the system from time to time. Smaller mains may have a copper blow-off pipe attached to the pipe controlled by a gate valve.

Trunk water mains require air relief valves. These valves are generally designed to operate automatically, permitting bleeding of air from the main whenever necessary. Air build-up will take place at a high-point in the system, as the velocity in the main will be at its lowest thereby allowing the air to gather at this high point. This can be considerable if the high point is located a long way from the reservoir if a gravity system or from the pump if a pressure system. If enough air collects without relief, a pressure block will form in the main severely disrupting supply.

In certain systems air valves can be installed which also act as vacuum valves. In this condition, the valve permits the flow of air into the line during draining of the line and the internal pressure reverts to atmospheric. This prevents a vacuum forming in the line and protects the system from collapse.

Drain Valves

Larger diameter watermains have drain valves installed at the lowest points of their alignment to permit draining of the main. The drain valves are gate valves generally placed on a tee to the side of the main, draining to a watercourse or other means of removal of the unwanted water.

Valve Chambers

These are generally provided for large valves which may have a handwheel for their operation or are electrically operated. The chambers may be poured-in-place or formed with precast sections. In either case, the chamber should be large enough to permit easy access to the valve both for operation or removal. Large valves may require couplings to be placed in the pipe beside them in the chamber to permit removal of the valve.

Valve Boxes and Stems

These are used for smaller valves where it is necessary to provide access to the operating nut. This access is provided by the use of a valve box and stem. The stem is a steel cylinder that is locked over the operating nut with 90° opposite set screws. The valve box is placed over the valve stem and provides access from the ground surface. The stem and the box assembly should be adjustable to permit adjustment with respect to roadwork and operation of the valve.

Hydrant

Fire protection manuals suggest that hydrants be located so that no building is more than 120m from a hydrant. Hydrants are equipped with a shutoff valve between the hydrant and the main. This facilitates the maintenance and replacement of hydrants without disturbance to the flow in the main.

Hydrants must be installed in a way to ensure that they do not move or that joint separation is caused by the water pressure. This is usually accomplished by the use of wooden or concrete thrust blocks at the base of the hydrant stand-pipe or by the use of hydrant anchor tees.

Service Connections

Service connections can be provided by the use of copper, cast iron, ductile iron or plastic pipe. The most common material used is copper, soft type K. The size more generally recommended is 20mm diameter. For 19, 25 or 50mm services, use copper, soft type K materials.

These services should be equipped with a corporation stop and a curb stop. The corporation stop is attached and threaded into the wall of the watermain. Consideration should be given to the thickness of the pipe wall and the tap size to ensure a serviceable, threaded connection. The corporation stop has a shut-off so that the stop may be set in the line under pressure and be subsequently opened for service.

The curb stop is normally situated at the property line and provides a shut off in the event that maintenance work is required on the private service. The curb stop consists of a valve, operating rod and box similar to that on a main line valve.

All copper water services should be installed with a vertical goose neck at the main to reduce the possibilities of ground settlement shearing the copper pipe from the corporation stop when tapped at 45°.

All plastic water services should have a horizontal goose neck when tapped at springline.

Bends, Wyes and Tees

Bends, wyes and tees are known as specials or fittings and are integral parts of a distribution system. It is at these locations that unsupported thrusts can develop and must be counteracted in the design. The inspector should ensure that pipe alignments are accurate at these junctions or changes in direction as leakage or even failure problems can occur if a joint is improperly made to account for an excessive angle developed by a poor alignment.

Thrust Blocks

The majority of pipes used for watermain work have joints which are not restrained longitudinally. The pressure in the watermain exerts a force on the pipe at bends, closed valves, tees and hydrants. Good thrust blocking practice is important to reduce breaks and leaks caused by these forces. Anchorage may be required in a vertical as well as a horizontal direction.

Thrust blocks are, therefore, installed on the outside of bends and tees to counteract these forces and can be formed of concrete in a manner that will transfer the forces to firm surrounding ground.

Thrust is only a problem at valves where there are couplings as the couplings cannot transfer the thrust longitudinally along the main. Generally, there is a valve chamber at these locations which is designed as a thrust block.

BEDDING

As discussed earlier, the route of a watermain is not necessarily a straight line and the conduit may be designed with a minimum cover. It is reasonable to expect that a contractor will use a minimum of trench sheathing material in his watermain construction except for larger pipes either with deep and wide trenches or located in a roadway. Under these conditions, excessively wide trenches or slash trenches might occur and cause problems with excessive loading on the pipe. A slash trench could have a slope of greater than 1 to 1. The trench bottom must first be drained, cleared of all stones and lumps and properly prepared to accept the bedding. The material under the pipe, generally crushed limestone or even clear stone should be graded and compacted to the desired thickness. The bedding must support the pipe over its entire length. Special care must be taken at the joints to ensure that the pipe does not rest on the bells of the joints alone. After the pipe has been placed and jointed, the sides should be backfilled and properly compacted to prevent lateral movement of the pipe and then carefully backfilled with granular material to about 300mm above the pipe before further backfilling proceeds.

MATERIALS USED IN WATERMAIN CONSTRUCTION

There is a good selection of pipe materials available for construction of watermains. Some are for specified applications, others are satisfactory for general use. The most common materials in use are:

1. Ductile Iron
2. Cast Iron
3. Reinforced Concrete
4. Steel
5. Asbestos Cement
6. Pressurized Plastic

Factors to be considered in the selection of the pipes are:

1. Cost
2. Availability in size and class
3. Ease of laying and jointing
4. Structural capacity in place
5. Facility for thrust restraint
6. Corrosion resistance
7. Bedding requirements
8. Types of joints

The most common joints used in pipe laying are:

1. Mechanical joints
2. Push-on joints
3. Field Welding
4. Victaulic couplings.

CONSTRUCTION CONSIDERATIONS

There are certain important points that the inspector should pay particular attention to during construction. Some of these are as follows:

1. The bedding must be to grade and a hole dug out for the bell to prevent bridging of the pipe from bell to bell on the bedding.
2. Final adjustments may be made to the pipe grade by lacing pieces of wood under the bell or front edge of the spigot piece but these must be removed when the grade is set.
3. The trench width should not exceed that shown on the drawings.
4. The bedding must be compacted under, around and over the pipe.
5. Large boulders must not be included in the backfill.
6. Compaction methods must ensure that all voids caused by removal of sheathing are filled.
7. Thrust blocks must be taken to firm, undisturbed ground.
8. Valve nuts must be kept clear of backfill material, and placed in a vertical position.

SUBJECT:

CONSTRUCTION OF SEWERS WATERMAINS AND ASSOCIATED FACILITIES

TOPIC: 16

CONSTRUCTION SURVEYING

OBJECTIVES:

The trainee will be able to:

1. Prepare Grade Sheets for the layout of sewers.
2. Recall the role of the inspector during construction with respect to layout.
3. Recall some of the various methods of Construction layout.
4. Better understand basic surveying.
5. Calculate end areas by datum line method.
6. Calculate volumes by average and area method.

CONSTRUCTION SURVEYING

INTRODUCTION

Surveying has been described as the science and art of making the measurements necessary to determine the relative positions of points above, on, or beneath the surface of the earth, or to establish such points.

Thus, within this broad definition, all of the following fields are included:

Legal Surveying
Topographical Surveying
Mine Surveying
Hydrographic Surveying
Geodetic Surveying
Photogrammetric Surveying
Quantity Surveying
Construction Surveying

Although construction surveying was listed last, it is the most important to the inspector involved in the installation of municipal services such as sewers, watermains, and roads. He should be familiar with all related surveying theory, background, current construction practice such as Grade Rod Theory and Transfer of Grades, in addition to comprehending contract specifications and requirements, and some appreciation of the design considerations related to the facility being installed. The question is how much of this background understanding of design, specifications and surveying should the inspector be familiar?

It is difficult to give a precise answer. A general answer would be that an inspector should have enough knowledge of design and surveying so that he can properly perform his duties.

Usually, but not always, an inspector from a smaller municipality has a greater over-all knowledge of a project than his counterpart in a large municipality. It seems that the larger an organization grows, the more specialized become the various jobs. However, an inspector should not be so specialized that he has no knowledge of the problems involved in construction layout and grades.

The basic construction team for a municipality consists of the engineer, designer, surveyor and the inspector; often the inspector and the surveyor may be one and the same man. The inspector is the last link in the chain of personnel responsible for the correct construction of a project. He is often the last man capable of detecting an error - either in design or layout. Therefore, if he has a good over-all knowledge of his job, he will be able to spot discrepancies that may arise, and in doing so, avoid the waste of materials and money.

A good example of a possible source of error is in the level shot, when the instrument man makes an error in reading or recording a rod shot on a grade stake. It is impossible to check, but there is a good possibility that the error will show up on the grade sheet. A capable inspector may detect such an error by noting that the erroneous grade does not fit in with the grades adjacent to it. Most surveyors, in fact, try to establish their layout in such a way, that possible errors are more easily detected by themselves or by the inspector.

The division of responsibility for survey work varies depending on the job specifications. The survey work may be done in any one of the following methods:

1. The Contractor undertakes all the layout.
2. Either the Contractor or the Surveyor carries out the layout, and the other party checks it.
3. The Surveyor provides the layout.
4. The Inspector does the layout.

Under alternative (2) it would not be necessary for both parties to conduct the complete survey, but rather certain check points would be selected such as Bench Marks and manhole stakes, and these would be 'shot' with an instrument. Comments which will be made later concerning 'checking' may be taken to apply to the Inspector checking the Contractor's layout or to the Inspector checking his Surveyor's layout.

Under no circumstances should the contractor's instrument man and the inspector work on the same survey team. All checks should be done independently.

CONSTRUCTION SURVEYS

The Contract Drawings will provide sufficient information to properly layout all the indicated services both in terms of Horizontal Orientation and in Vertical Relationship to the surface of the ground.

Horizontal control is provided by means of:

1. A legal survey plan, showing dimensions and bearing of property boundaries in a subdivision.

2. A centre line alignment drawing, giving tangent lengths and azimuths as well as appropriate circular and spiral curve data sufficient to lay out services.

3. A co-ordinate grid system, having key control points identified by co-ordinates, which are then used to locate other construction points.

Vertical control is usually provided by bench marks related to the basic network of high accuracy bench marks established by the Federal Government.

The Contract Drawings will always show a "Site Bench Mark" which will be related to either Geodetic Survey Datum or Hydrographic Survey Datum. This may, in fact, be some distance from the sewer to be installed and the Inspector will then have to transfer his bench mark to within 60m of the start of the sewer and thence establish temporary bench marks (T.B.M.) at each street intersection, or at approximately 150m intervals to the end of the job. This system of T.B.M.'s would then be looped back on the original bench mark. After all of the T.B.M. levels have been reduced and checked they may be made available to outside parties.

During the detailed design, the designer selects the sewer locations and elevations with the following points in mind:

1. The ground over which the sewer passes must be within the jurisdiction of the sewer owner, either under direct ownership or by means of easements or lease agreements.

2. There must be no conflict with existing buildings or underground pipes.

3. The gravity sewer must be high enough to drain into the top end of the existing system.

4. The sewer should be low enough to drain all house basements.

5. The slope of the sewer must be sufficient to provide a flow velocity of at least 0.6m/sec with the sewer flowing full.

Preparation For Surveying

Prior to any surveying being carried out, the surveyor should be fully briefed by the resident engineer or inspector as to the specific stakeout task, the acceptable tolerances, the expected time to completion, and any other information that would affect the stakeout work. This is an important point, as a misunderstanding of the work to be carried out can lead to a loss of time by the field party, a loss of production by the contractor and a general lack of confidence and co-operation among all concerned.

The stakeout of the various services to be constructed, together with the contractor's ability to follow accurately the field lines and grades, determine the ultimate conformity of the completed works with the design specifications and drawings. Obviously any errors on the part of the field personnel, if not detected will have serious repercussions.

It is therefore essential that all stakeout work be independently checked for line and grade before grade sheets are completed and handed to the contractor.

Before commencing a stakeout, the surveyor should have in his possession the following:

1. Field job file, containing a complete set of "Approved for Construction" drawings for the project under construction, list of local bench marks, copy of an up-to-date registered plan, and any previous relevant field notes.

2. A complete set of equipment in good adjustment, including notekeeping equipment.

3. Written instructions, when possible, outlining specifically the work to be carried out.

The first thing to do upon arriving at the work site is to obtain an overall view of the project, with the intention of determining the most efficient way of completing the job, i.e. accurately in the shortest time. Study the plan for a few minutes in relation to the surrounding area, noting any unusual features such as services not in normal location, flow directions, physical obstructions to layout, and so on. It has been proven many times that a little time and thought spent at the beginning of a stakeout, can save hours of time lost, when work had to be re-done because of the lack of planning or a full understanding of the task.

Construction Layout

Once the Surveyor has familiarized himself with the work, he can then proceed with the layout of the sewer, or whatever, by determining the location of the manholes from the contract drawings in relation to known points such as iron bars.

If there are several manholes in a straight line it may be advantageous to stake the first and last manhole in line. Using a right angle hand prism, or for greater accuracy a transit, two offset stakes are established on the side of the sewer line opposite to that on which the backhoe will cast the excavated material. For shallow sewers the offset would be 4 or 5m increasing to 7 or 8m for large, deep sewers.

Stakes are usually 50mm x 25mm x 450mm rough lumber pointed at one end. They must be driven down accurately as the exact offset line is marked on the top of the stake, with a 25mm nail driven part way into the stake to indicate the offset line. Stakes must be marked with the proper chainage, offset distance, direction, and preferably the nature of stakeout (i.e. san or storm) to avoid confusion by contractor's crews.

Intermediate stakes are then driven in at 10m intervals starting with Sta. 0+00 at the M.H and finishing with the next M.H. A nail is located on line at the top of each stake (see Figure 16-1).

When laying out a sewer the surveyor can start his chainage (stationing) at the low end of the work and then continue it through several manholes to the upper limit of the job. However, the surveyor usually begins a new chainage at each new manhole. In this case each new manhole, or existing one (if the project begins at an existing manhole), has an initial chainage of 0+00.

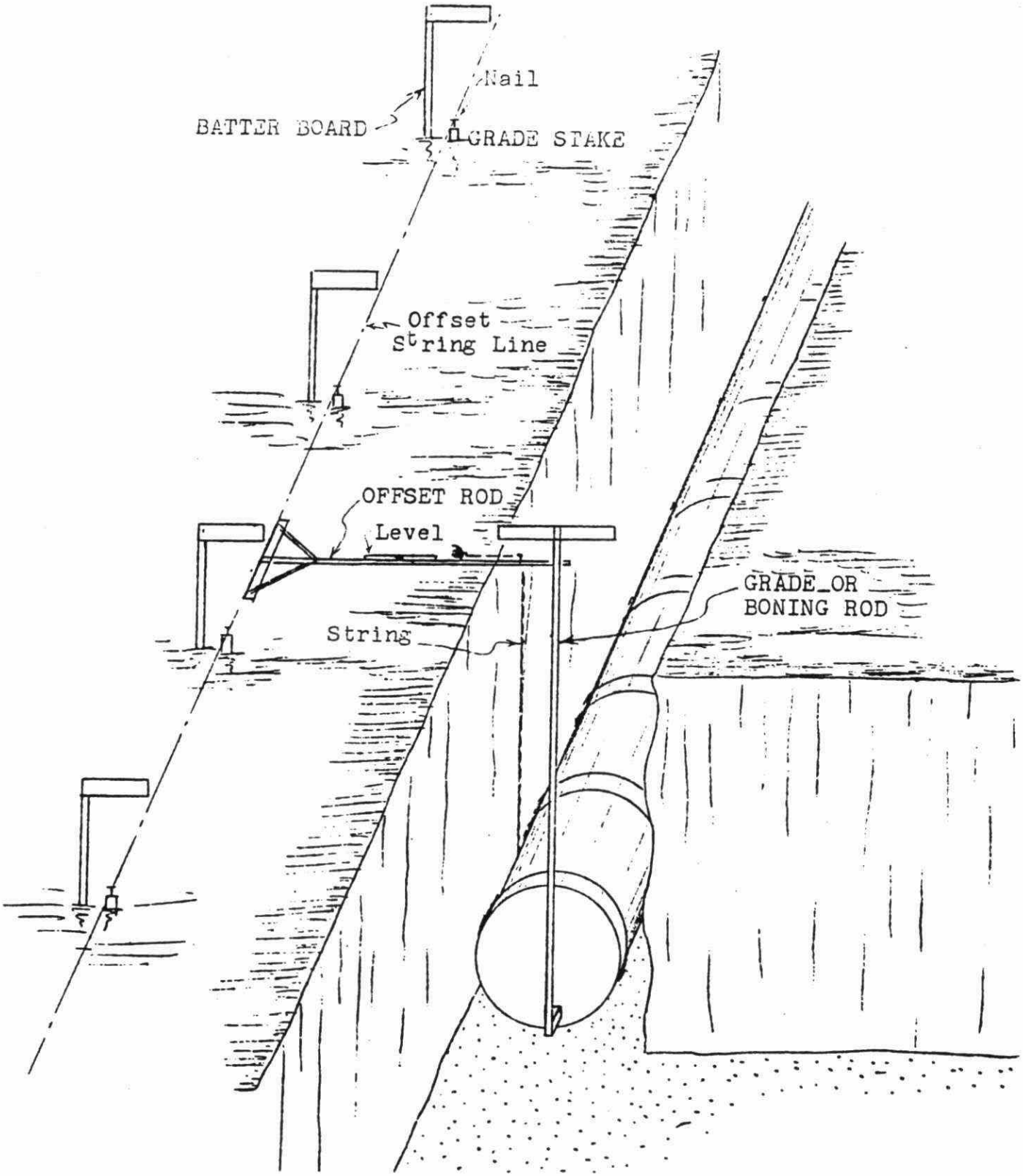
As soon as the stakes have been laid out the Contractor must be informed and instructed to protect them by means of larger stakes or boulders so that they are not disturbed by trucks, bulldozers or children.

In all cases when extending or connecting into an existing line, the offset at the appropriate end must be taken from the existing service, and not from plans. (e.g. when extending an existing line the offset must be taken from the sewer line and not from the manhole cover.

At the time of stakeout, the Contractor should expose the end of an existing service in order that the surveyor can confirm the elevation and location of the existing invert.

On standard subdivision roads sanitary and storm sewers usually run parallel and 3m apart being respectively 1.5m from centreline. Where both manholes are on a straight portion of the road, the manhole at the low end of the line should be located first, and offset at right angles to the centreline.

Figure 16-1


Nail
BATTER BOARD
GRADE STAKE
Offset
String Line
OFFSET ROD
Level
GRADE OR
BONING ROD
String

Where starting a line at a street intersection, or at any location at which the '0+00" stake cannot be set or may be disturbed, a '0-5' stake should be placed to provide the contractor with a sight beyond the end point. The distance to the minus stake may of course be varied to suit local conditions.

Distances from manhole to manhole are given on the construction drawings and usually represent scaled distances. It is advisable to check the given distances with a scale before accepting them, or, to compare the individual distances added together on a given street with the total chainage of the same, making allowances for intersections. If no errors in distances are apparent, the distances should be kept as shown on the drawing. The terminal manhole is to be located prior to stakeout of the offset line and the actual chainage determined by measurement on a straight road between intersections. The distances are calculated and any discrepancy in the last length can only be due to inaccurate chaining, but on a curved road with scaled distances, a discrepancy of a metre of so can be accepted as normal.

Locating manholes at irregular intersections or on curved portions of a road is easiest done by scaling the distances from the two or three nearest survey bars and intersecting the arcs of respective scaled values. The intersection angle should be as close to a right angle as the situation permits. Having located MH.A this way, MH.B, can be located also by scaling the distance of centreline of the sewer from the nearest B.C. or E.C., and projecting it by the scaled distance to MH.B. Having done this, we can offset from both manholes and establish the offset line. The actual distance measured should agree reasonably well with the distance given on the drawing.

Situations are common where the party chief cannot follow any fixed procedure in laying out sewer lines and must depend on his own best judgement in choosing the method that guarantees the closest possible adherence to the design. Often it is necessary to re-establish the road allowance and the nature of the available evidence suggests the method for doing this. Special care should be taken when a manhole on a curved portion of road is located near the curb.

As the stakeout progresses, final tie measurements should be made from the proposed manhole locations to any adjacent survey bars, existing manholes, etc., to ensure that no blunders have been made. These should be made on tracing paper of field note size as shown in the sample field notes, so that the inspector can check the stakeout in the office.

On every lay-out job checking one's own work is never a wasted effort though in 99 times out of 100 it may appear so. There is an infinite variety of possibilities for errors and the best policy for the party chief is to establish a routine whereby no essential step in his lay-out work is left unchecked. Not only the actual measurements deserve attention, but also the basic information from which he started out deserves a second scrutiny. Superficial digestion of information given on drawings may result in serious errors. It is brought to the reader's attention that the Laser Beam is currently being used for construction layout purposes, however a full discussion on its use is included in Appendix 16E.

Construction Grades

Levels to establish grades for the various services are taken on the grade stakes. New road men should be strongly reminded to take elevations on the top of the stakes, not on the ground. Levels are normally taken to the nearest 5mm and must be originated on one bench mark and terminated on another. Elevations of existing services, to which services under construction are to be tied, must be taken, even if the existing services were only recently installed.

If there is any doubt as to the elevation of a bench mark, i.e. vague source, signs of physical damage, etc., a check must be taken. All bench marks must be used as turning points.

When establishing bench marks, select only points which will remain undisturbed throughout the construction period, and don't hesitate to set more than you feel will be adequate. Bench marks must always be carefully described (i.e. "B.M. 5B" is not adequate). The notes taken during the setting of B.M. 5B may be lost, and then so is the bench mark.

All turning points and points sighted must be properly and adequately described. Stake stations must be carefully noted at the time of stakeout, i.e. don't level the grade stakes, then complete the notes in the car or office.

Field notes must be checked for closure, i.e. add back sights and foresights, before leaving the field.

GRADE SHEETS

A grade sheet is the tabulated form of providing to a contractor all the information pertinent to the installation of a particular facility. The grade sheet should enable the contractor to install the particular sewer, or whatever, as indicated on the contract drawings. Grade sheets are necessary for the installation of a variety of different services including:

1. Storm and sanitary sewers
2. Lateral service connections
3. Water mains
4. Water services
5. Curbs (but usually, not sidewalks)
6. Catchbasins
7. Road grading and sub-grade.

The information contained on a sewer grade sheet includes:

1. Project name
2. Date
3. Contractor's name
4. Type of sewer - sanitary or storm
5. Location of the sewer line, as well as the upstream and downstream manhole numbers
6. Sewer size, type, grade bedding an backfill
7. Location of the lateral connections - chainage and direction

8. Manhole number, type, depth and relationship of the inverts w.r.t. the lowest invert
9. Signature of the person who computed that grade sheet
10. Layout stations, stake elevations, invert elevations, cut depth and measurements to the batterboard.

Grade sheets are generally made out by the inspector, but if this is not possible, they should be computed by the surveyor who carried out the stakeout. However, here are some general observations to bear in mind.

1. The grade sheet must be accurate as an error may mean a very expensive mistake in construction.
2. Completing the grade sheet is the time when the inspector will probably look at the drawings in the greatest detail, so he should take this opportunity to check if there are any inconsistencies or mistakes on the drawings which may require clarification by either the design engineer or the supervision engineer.
3. The inspection must realize that the construction foreman who receives the grade sheet may not have seen the drawings, so a note or diagram should be on the grade sheet, drawing the foreman's attention to anything unusual such as a drop connection or a special type of manhole cover.

Cut Determination

Usually the slope of the sewer is given but sometimes it must be calculated. From this slope, the sewer invert elevation at each of the grade stakes is found. Occasionally, the top of the pipe is the reference point rather than the pipe invert. The reference point must be clearly indicated and described. Normally, slope is given as a per cent.

i.e. $\% \text{ Slope} = \dfrac{\text{Difference in Elevation}}{\text{Horizontal Distance}} \times 100$

Therefore a 2.5% slope $= \dfrac{2.5 \text{ m}}{100 \text{ m}} \times 100$ (the slope is 0.025m per linear metre)

To calculate the invert elevation at chainage "X", take the invert elevation at Sta. 0+00 and add to it the slope times the distance to chainage "X". This is taken to two decimal places. An electronic calculator can be used to considerable advantage in these calculations.

At this point it is wise to remember that all calculations should be checked. If you are subtracting B from A to get C then add B and C to see if you arrive at A again. By the same token, invert elevations can be derived by a process of repeated additions.

When the elevation of the top of the offset stake has been determined, and when the elevation of the proposed sewer or structure is known opposite that station, then we can calculate how much we must measure up or down from the top of the stake to be at the proposed elevation of the structure at that point.

Example: We know that the elevation of a proposed sewer invert at a particular point is to be 125.00. The surveyor determines the top of stake elevation to be 127.44. We now know that if we measure 2.44m down from the top of the stake that we will be at the proposed elevation. We call this a cut of 2.44m (if the stake had been below the proposed elevation, we would have to measure up, and in that case we would be talking of a fill of x metres. These cuts and/or fills are entered on the grade sheet which is then given to the Contractor.

Grade Transfer

The next point to consider is how to transfer the cut, or fill, dimension from the top of the stake across the offset distance to the proposed construction. This can be done directly where the offset is small by using a carpenter's level and a straight edge; however, where the offset is more than 1.5m the transfer is done by means of "batter boards", (see Figure 16-1).

Most text books suggest that batter boards be placed across the trench with nails driven in the top to hold the centre line for the pipe. However, with the advent of extremely efficient excavating machinery, that method has become obsolete. Batter Boards are now located adjacent to the grade stakes on the opposite side of the trench to the excavated material.

A batter board is a 50mm x 100mm post placed firmly in a vertical position near the grade stake. A cross-tee is nailed to the batter board at a specified distance 'H' above the top of the grade stake. The distance 'H' is different at each grade stake and is calculated so that all the cross-tees are parallel to and some even number of metres above sewer invert.

For example, if the top of the grade stake represented a cut of 480mm, the Contractor would make H equal 250mm, 450mm, 750mm, etc. depending on what grade line fitted best with the other grade stakes. If the Contractor made H equal 250mm he would have a reference grade line 730mm above his proposed grade line - if he made H equal 450mm, the reference line would be 930mm above his proposed grade line. The important thing to note here is that although the H dimension would be different for each stake, the cross-tees would all be the same distance above the proposed sewer grade at each stake.

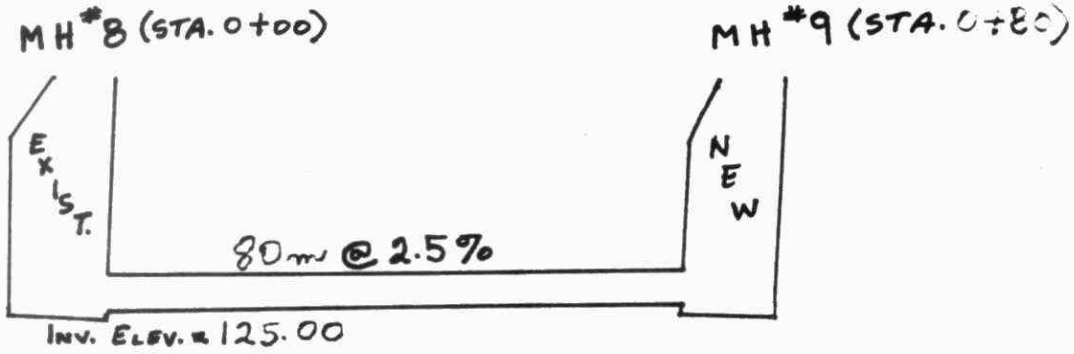
Once he has all his cross-tees set properly, the Contractor can sight down from the tees and check levels at any point along the sewer line.

In the case of a fill grade, say 480mm, the Contractor knows if he measures up the batter board stake 480mm that he will be at proposed grade, however, this would leave him little room in which to work so it is usually found that the Contractor will utilize a vertical offset of at least one metre. In this case, he would actually measure up the large batter board a minimum of 520mm and so have a constant cut measurement of at least one metre from his level line of sight on the cross-tee.

A grade rod, or boning rod, is a measuring pole constructed so the distance from a cross piece at the top to the bottom of the pole is the same as the distance between the set batter boards and the proposed grade. If the Inspector sights over the batter boards and finds that the cross arm of the grade rod lines up exactly with the plane of the tops of the batter boards, he knows that the boning rod is on grade. If it doesn't line up, the necessary adjustment up or down is made in the trench. A minimum of 3 batter boards is necessary for any one accurate sighting. Sometimes additional cross pieces can be fixed to the boning rod to indicate trench bottom or a second pipe.

The length of the boning rod and the height of the batter boards is determined by the depth of cut. The plane of the top of the batter boards must not be so low that the Inspector has to lie on his stomach to sight them, or so high that the line across the boards is over his head.

Example: It is proposed to extend an existing sanitary sewer 80m. We know that the invert elevation of the existing manhole (M.H. #8) where we will make the connection is 125.00. Also based on the various design parameters, the sewer must be installed at a grade (slope) of 2.5%.



Let us now prepare the Grade Sheet for laying this sewer out.

1. It is necessary to set out an offset line with grade stakes at 10m intervals along it. Use the existing M.H. #8 as Sta. 0+00. This makes the new M.H. (#9) Sta. 0+80.

2. By measurement, we find that the elevation of the top of each of the grade stakes along the offset line is as given in the column "Stake Elevation" on the Grade Sheet (Figure 16-2).

3. Determine by calculation the invert elevation of the sewer opposite each of the grade stakes (Sta. 0+00, 0+10, 0+20, etc.). This is done by using the following formula, i.e. Invert Elevation at Sta. X = Invert Elevation @ Sta. 0+00 + Slope times the distance X.

Example:

$$\text{IE Sta. } 0+30 = \text{IE (Sta. } 0+00) + \frac{2.5}{100} \times 30$$

$$= 125.00 + 0.75 = 125.75$$

OR

$$\text{IE Sta. } 0+10 = 125 + \frac{2.5}{100} \times 10 = 125.25$$

Using the above formula, the Elevation given under the column "Invert Elevation" on the Grade Sheet (Figure 16-2) were found.

4. By calculation we determine the difference in elevation between the sewer invert and the top of the grade stake at each of the Stations. This is the information given in the column "Cut" on the Grade Sheet.

5. By trial and error determine the most suitable grade rod length. If we try 3m at Station 0 + 08, the top of the batter board will only be 525mm above the stake - this is too low. If we pick a grade rod of 5m we would be 2.575m above the stakes with the batter boards - this is too high.

Rule of Thumb: The grade rod is obtained by taking the greatest figure in the "cut" column and adding 1 metre to the next highest metre, i.e. if the greatest figure in the "cut" column is 2.475m; the length of the rod could be 4m.

Use grade rod length of 4m for this example.

6. To determine how high above the top of the grade stakes we must measure to have a line of batter boards parallel to and 4m (the length of the grade rod) above the sewer invert, subtract the "Cut" from the grade rod length. This information is given in the left side of the column "stake to Batter Board". The right side is the conversion to millimetres so the contractor can easily set up his line of batter boards.

GRADE SHEET

CONTRACT NO. .....

CONTRACTOR.....

Layout by..... Book No. ..... B.M. .....

Gradient ..... 2.5% ..... Offset ..... Grade Rod ..4m..

Station		Stake Elevation	Invert Elevation	Cut	Stake to batter board	
Backsite	0-10	127.410	124.750	2.660	m 1.340	mm 1340
M.H.8	0+00	127.460	125.000	2.460	1.540	1540
	0+10	127.650	125.250	2.400	1.600	1600
	0+20	127.585	125.500	2.085	1.915	1915
	0+30	127.960	125.750	2.210	1.790	1790
	0+40	128.105	126.000	2.105	1.895	1895
	0+50	128.180	126.250	1.930	2.070	2070
	0+60	128.355	126.500	1.855	2.145	2145
	0+70	128.540	126.750	1.790	2.210	2210
M.H.9	0+80	129.475	127.000	2.475	1.525	1525
BEDDING		TYPE B				
LOCATION OF TEES						
RIGHT		0+25	0+40			
LEFT		0+10	0+30	0+70		

Figure 16-2

Alternative Methods of Grade Transfer

As an alternative to batter boards but less accurate and not recommended for general use are the Carpenter's level, String level and Hand level. All these contain a liquid with an air bubble in a curved tube which centres itself in the tube when the instrument is level.

1. Carpenter's Level

   This level is mounted on a piece of wood or metal between 150 and 900mm long. The Carpenter's level can be placed on top of a straight edged piece of lumber to transfer grades up to 4m with an accuracy of 3mm.

2. String Level

   This level is supplied with a hook at each end so that it can be hung on a string which is then stretched from the point of known elevation such as the grade stake to the centre of the work area where the elevation is required. The string level should be at the centre of the string so as not to be upset by the curve or catenary of the suspended string. The accuracy is 16mm in 1m.

3. Hand Level

   This is a simple hand held tube with a 45° mirror trained on the bubble and located in the line of sight. The tube is open at each end and there is a hair line to indicate the axis of the tube. The tube is rotated vertically until the bubble comes into view and is bisected by the line of sight is horizontal. An observation may then be made of point of equal elevation, since no magnification is provided the viewing area is limited to about 15m. This method is the least accurate of any of those mentioned being only about 20mm in lm.

House Connections

Tee connections for house service laterals should be located by stakes along the offset line. It is common practice to use the main elevation stakes for this purpose so that instead of an even 10 or 20m interval between stakes, the interval will vary to suit the house services. This means more work when calculating the grade sheet but less work on field layout.

In new subdivisions there will be little disagreement on the position of the house service. The builder will provide a plot plan and will ask for specific service locations for each different type of house in the subdivision.

In built-up areas the policy of each municipality will vary. It is customary to consult with each house-holder to find out where they would like the service brought in. This discussion can be abbreviated if a wooden stake is left with each householder, and he is asked to drive the stake at the point on the street line where the service is to terminate. The main sewer tee is then located opposite this point on the main sewer.

In extreme cases, and in order to avoid misunderstanding, the Municipality may draw up a "House Service Location Sheet" showing the service location as requested by the home owner. It is signed by the owner and copies distributed to the interested parties including the Inspector and the Contractor. This procedure is fool-proof but rather time consuming.

Checking of Layout

The layout is now complete, but the first sod has not yet been turned, and the Inspector's job is only just beginning. Remember that the Inspector is the last vital link in the design-supervision chain.

You will notice that the first stake on the grade sheet is at chainage 0 - 10. This stake and batter board may be erected where the manhole is being excavated. Before any pipe is laid the next two batter boards must be erected and all three checked to see that they are in line. A minimum of three batter boards must be in position at all times. In this way most errors in layout, levelling or calculation may be detected immediately. An error in selecting the sewer slope will not show up in this way.

The boning rod should then be made up with the toe well braced in position at right angles to the shaft, and the cross-tee firmly nailed in position at right angles to the shaft in the other direction. Measure the distance from toe to cross-tee to see that it agrees with the grade sheet.

Now proceed to the batter boards and check the dimension "stake to batter board" against the grade sheet and set a hand level on top of the batter board to see that it is truly level, if it is off, correct it and check the distance down to the stake.

Since the batter board does not span the trench a small variation from level will throw your line of sight off considerably.

Place the boning rod vertically with toe in pipe invert. Always sight in the same direction, preferably in the direction of pipe laying. The boning rod tee must be in exactly the same plane as the three batter board tees.

The Contractor must set a string line between the first and third grade stakes, it will be fixed to the nail on top of the two stakes mentioned and should pass right beside the nail in the intermediate stake. The string line should be on a minimum of three stakes at all times.

The Contractor should then make up an offset rod with a small right angle tee at one end. Two nails are placed along the rod at the correct offset distance from the tee and a plumb bob and string hung down. On small pipe the plumb bob will be situated at pipe centre line, on 600mm diameter pipe and larger the offset measurement on the rod is often reduced by half the pipe O.D. so that the plumb bob marks the outside edge of the pipe barrel. In either case, the Inspector must check the offset rod length from tee to plumb bob nail and also see that the tee is held gently against the string line without moving the string.

Finally, the Inspector enters the trench and checks to see that the pipe lines up with the plumb bob.

QUANTITY CALCULATIONS

During the construction of a project, it is often necessary to calculate volumes of materials - the excavation of roadways, the excavation of sewer trenches, the amount of material removed from a borrow pit and so on.

Although there are a number of ways of computing volumes, one of the most common methods used which is simple yet accurate is the Average End Area Method. There are available a number of sophisticated computer programs which are based on this method and which can provide a wide variety of output data. We will at this time, only attempt to describe the approach to calculating quantities and direct the student to other, more advanced references, if he is interested in pursuing the computer approach.

The quantity calculation procedure by the average end area method is summarized as follows:

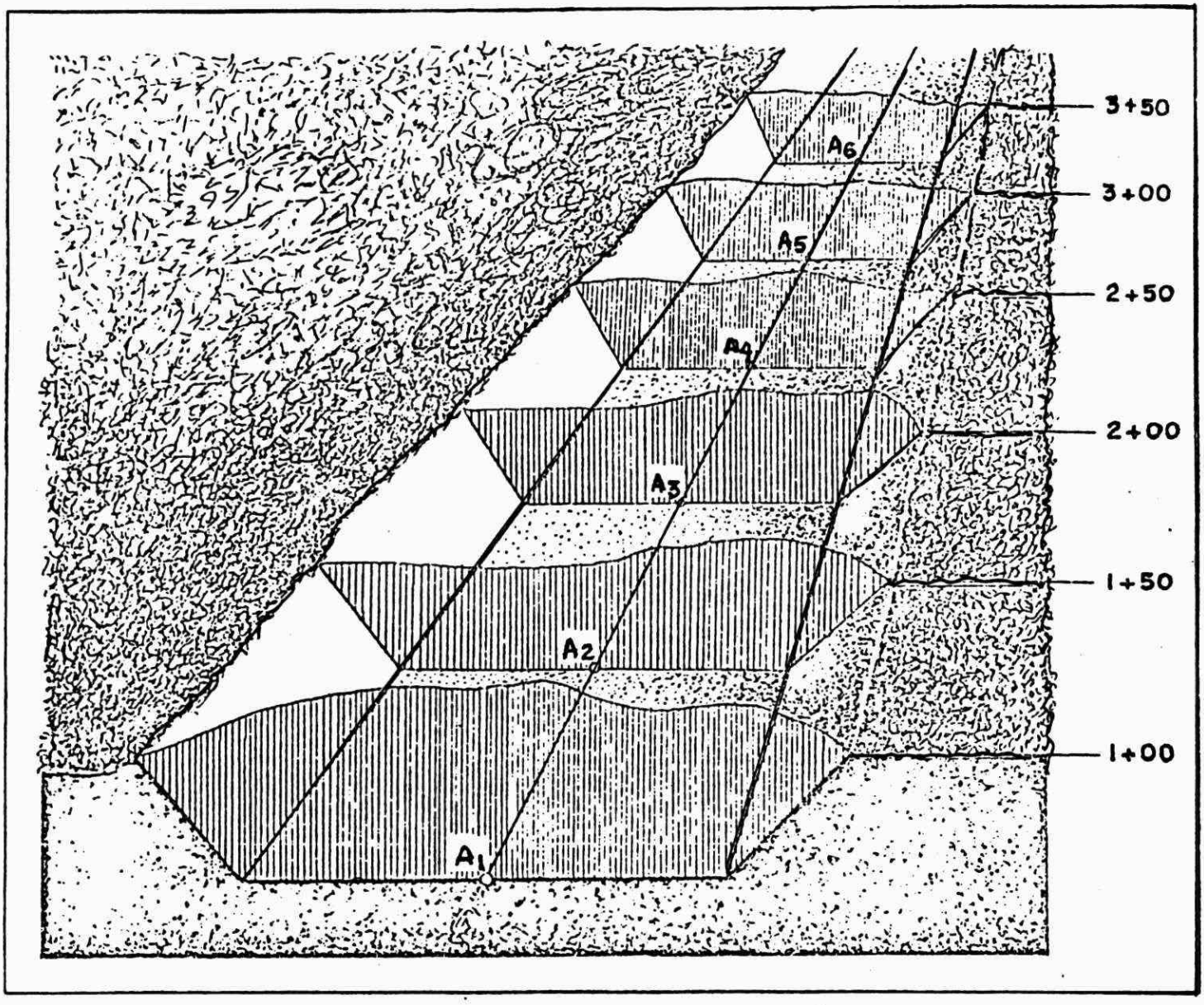
1. Take, original cross-sections of the area to excavated at suitable intervals.
2. After excavation, take the final cross-sections at the same locations as the original ones.
3. Plot both cross sections to scale and compute the difference in end area at each station by the Datum Line Method.

4. Having obtained the end areas at each section, set up the "summary quantity sheet" according to chainages as shown. The calculations using the mean distance between chainages and the end areas calculated earlier, will give the total volume excavated.

As can be seen, this principle can be readily adapted to compute sewer trench excavations, as the cross-section would be considerably simplified.

Figure 16-3

ILLUSTRATION OF THE THEORY OF VOLUMES From AVERAGE END AREAS



END AREA COMPUTATION BY DATUM LINE METHOD

1. Plot the cross sections (including the originals and finals) showing the elevations and offsets to the Department Standard.

2. Choose the next tenth datum line below the lowest point on the section. Any datum below the lowest point could be used but for ease in calculation the even ten is commonly used.

3. Average the depths to the datum line for each pair of points in succession on the original ground line

4. Multiply the average obtained between each pair of points by the distance between them.

5. Add the products obtained to get the area between the original ground line and the chosen datum line.

6. Repeat the same calculations using the final line and the same datum line. This will give the area between the final line and the datum line.

7. Subtract the final area from the original area to obtain the area of the cut section.

8. If fill section areas are required the same method may be used except that the original area be subtracted from the final area.

9. These calculations should be entered on Department of Highways form OB-C-3 and OB-C-4 as shown on the following pages.

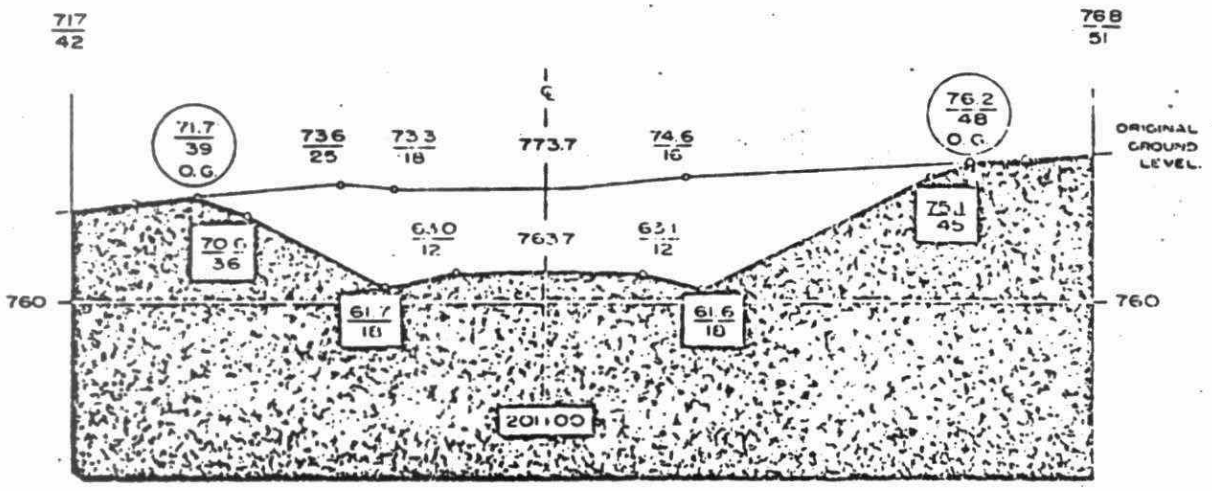


Figure 16-5

MINISTRY OF TRANSPORTATION AND COMMUNICATIONS

ONTARIO

CROSS-SECTION SHEET

SHEET 9 OF 14 DATE August 8th 1973

CONTRACT NO. 65 + 122 ITEM NO 3 EARTH EXCAVATION

LOCATION OF MATERIAL ETC In Road Cut From Station 200 + to 203 + 80

STATION	PLUS	SUB-AREA	MINUS	SUB-AREA	END AREA
201+00	$\frac{(11.7+13.6)}{2}\times 14$	177.10			
	$\frac{(13.6+13.3)}{2}\times 7$	94.15	$\frac{(11.7+10.6)}{2}\times 3$	33.45	
	$\frac{(13.3+13.7)}{2}\times 18$	243.00	$\frac{(10.6+1.7)}{2}\times 18$	110.70	
	$\frac{(13.7+14.6)}{2}\times 16$	226.40	$\frac{(1.7+3.0)}{2}\times 6$	14.10	
	$\frac{(14.6+16.2)}{2}\times 32$	492.80	$\frac{(3.0+3.7)}{2}\times 12$	40.20	
	87				
		1233.45	$\frac{(3.7+3.1)}{2}\times 12$	40.80	
			$\frac{(3.1+1.6)}{2}\times 6$	14.10	
			$\frac{(1.6+15.1)}{2}\times 27$	225.45	
			$\frac{(15.1+16.2)}{2}\times 3$	46.95	
			87		
				525.75	707 70
	(CHECK)				or 708
201+60	$\frac{(22.2+21.5)}{2}\times 8$	174.80	$\frac{(22.2+21.6)}{2}\times 4$	87.60	
	$\frac{(21.5+18.8)}{2}\times 14$	282.10	$\frac{(21.6+18.8)}{2}\times 8$	161.60	
	$\frac{(18.8+14.2)}{2}\times 14$	231.00	$\frac{(18.8+10.9)}{2}\times 12$	178.20	
	$\frac{(14.2+9.2)}{2}\times 28$	327.60	$\frac{(10.9+10.6)}{2}\times 28$	301.00	
	64				
		1015.50	$\frac{(10.6+9.2)}{2}\times 12$	118.80	
			64		
				847.20	168 30
	(CHECK)				or 168

DETAILED BY V Cosens CHECKED BY R. Flint SUBMITTED BY J. Helliot

NOTE:

1 — THE ITEM NUMBER IN HEADING SHOULD INCLUDE THE DESCRIPTION.

2 — THE ABOVE NOTED "CHECK" IS VERY IMPORTANT.

3 — SUB AREAS ROUNDED TO TWO DECIMAL PLACES END AREAS ROUNDED TO THE NEAREST SQ.

4 — USE THIS AS DECIMAL LINE.

5 — USE SIGNATURES AND NOT INITIALS

Figure 16-6

SUMMARY QUANTITY SHEET

SHEET ___ OF ___ DATE _____

DISTRICT No. _____ HWY. No. _____

CONTRACT No. _____ JOB NAME _____

LOCATION OF MATERIAL ROAD CUT STA 200+70 TO 203+80 ITEM No. 3 EARTH EXCAVATION

STATION	DIMENSIONS	DISTANCE	DOUBLE DISTANCE	MEAN DISTANCE	END AREA	CUBIC METRES
		0				
200+70	Start of Cut V.F.		30	15	527	7905
		30				
201+00			90	45	708	31860
		60				
201+60			80	40	168	6720
		20				
201+80			40	20	557	11140
		20				
202+00			70	35	1625	56875
		50				
202+50			100	50	1891	94550
		50				
203+00			130	65	2417	157105
		80				
203+80	Zero of Cut		80	40	0	
		0			Cu. M.	366151
203+80						
200+70						
310				310		
	(CHECK)					

DETAILED BY K Carson CHECKED BY R. Flint SUBMITTED BY J Lelliott

NOTE

- The Item No. in the Heading should include the number and description.
- The above noted "CHECK" is very important.
- Make this the Decimal Line.
- All areas to be taken to the nearest Sq. M.
- Detail Full Cut in Cubic Feet and then reduce to nearest Cubic METRE
- Use Signatures. (DO NOT PRINT)

APPENDIX - 16A

GENERAL SURVEYING

Surveying is an ancient art which can be traced to the earliest times. The Egyptians probably had the first well organized system of land measurement including angles and distances and the Romans must have credit for a sound understanding of the principles of the level as shown by the great aqueducts.

"Surveying" is a broad term and includes at least the following divisions:

Property or legal Surveying
Topographic Surveying
Construction Surveying
Mine Surveying
Hydrographic Surveying
Geodetic Surveying
Photogrammetric Surveying
Quantity Surveying

All of these are discussed in general terms in the Encyclopedia Brittanica covering five full pages.

The "Construction Surveying" portion of the Encyclopedia's reference covers only one tenth of one page. However, this is the area of prime interest to the construction inspector and is mentioned only to show that we are dealing in a highly specialized, or very narrow field, depending on the point of view.

REQUIRED MEASUREMENTS

Essentially and very simply all surveying is involved with only three forms of measurement:

1. Distance
2. Elevation
3. Angular Displacement

And, slightly oversimplified, a separate instrument is used for each as follows:

1. Distance
2. Elevation - Level
3. Angles - Transit

Each of these operations will be discussed, although in practice, frequently at least two and often three different measurements are being taken simultaneously.

SURVEYING EQUIPMENT

The list of today's surveying equipment includes transits, levels, tapes and chains, chaining pins, plumb-bobs, stakes, marking crayons and field books; all of which must be of the interest to the construction inspector.

In addition, the inspector must be aware that today's survey equipment also includes such highly refined apparatus as Shoran, Loran, and the Electronic Position Indicator, all of which use radar; the laser beam which uses visible light; the geodimeter which is a light-signalling instrument; and the tellurometer which uses radio microwaves.

APPENDIX - 16B

CHAINING

The Chain: A "chain" is a strip of steel approximately 6mm wide and 0.7mm thick and usually 50m in length.

A hundred years ago a chain was a series of links and was usually sixty-six feet in total length; the work "chain" has persisted, but fortunately we now work in 50m lengths.

Chains may be graduated in tenths, hundredths, and thousandths of a metre over the entire length or only in the first and last metre, with intermediate metre marks.

A "tape" is usually of cloth, fibreglass or steel and is not as accurate as a chain when used for cumulative measurements, i.e. running totals.

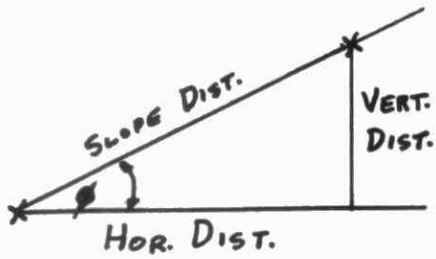
Corrections to Field Measurements

1. Chain Correction: A chain on tape must be compared with a standard to determine its actual length. For our purposes, a new chain is assumed to be exactly 50m long. Older chains and repaired chains should periodically be checked, using a new chain as a standard.

2. Slope correction: When a measurement is taken with the chain horizontal, there is no slope correction. Almost all measurements taken in municipal surveying are taken with the chain horizontal.

In the event that a measurement must be taken on a slope (i.e. not horizontal), the slope distance must be reduced to its horizontal equivalent. In order to do this, the angle of the slope with the horizontal must be taken. This angle can be taken with a transit or an Abney hand level.

Slope Distance	=	Distance Measured
Horizontal distance	=	Distance Required
θ	=	Angler Measured



$$\text{COSINE } \phi = \frac{\text{HOR. DIST.}}{\text{SLOPE DIST.}}$$

Horizontal Distance = Slope Distance x Cosine θ

As an example, a 50m chain held with one end 1m higher than the other will only give a horizontal measurement of 49.99m, an error of 0.01m, unless the slope correction is made.

3. Sag: If a chain is supported throughout its entire length (i.e. lying on the ground), a tension of 2.2N must be applied in order that the chain give a correct measurement. If the chain is entirely unsupported (i.e. supported only at the ends), a tension between 4 and 5N must be exerted to give a correct measurement. The range between 4 and 5N is to cover the different weights of chains now manufactured. The most common imperial unit chain in municipal survey use is 100 ft. long and requires a tension of 25 lbs. when supported only at the ends. Chainmen should find out the correct pull for the chain they are using and get the feel of this pull so as to be able to exert this tension when chaining.

4. Temperature Corrections: The coefficient of expansion of steel is 0.00000358 per degree Celsius. This amounts to about 0.002m per 50m per 10°C. A tape is standardized at 20°C.

   Example: If the temperature is 31°C, the chain would be longer by 0.002m. If you wanted to measure out 50m, under these circumstances you would measure out 49.998m.

Good Chaining Practices

1. Find the exact points that mark the ends of the chain.
2. Find by what method the chain is graduated and note how it should be read.
3. Agree on a set of signals with the other members of the party such as calling chain, mark, O.K., etc.
4. Find by trail what the correct pull on the chain feels like.
5. Make sure of the metre reading by checking the numbers on either side of it.
6. The head chainman should keep approximately in line (if a transit is not in use) by looking backward over the line.
7. Do not jerk the chain or pull on it when it is kinked, since it will easily break.
8. The rear chairman should not get in the line of sight when the transitman is lining in the head chainman.
9. The rear chainman should not carry his end of the chain between stations; he should allow it to drag free.
10. Do not allow the chain to be humped by a high ground level between each end of the chain.
11. Tie the thongs close to the eye of the chain and leave the ends of the thongs loose.
12. Use chaining pins or call stations every 50m.

Accuracy

Even experienced chainmen have difficulty in preventing the chain and plumb-bobs from moving during measurement. The error from this source is between 1:5000 and 1:10,000 with a chain of average cross-section, 50m long, supported only at the ends. Temperature may introduce an error up to 1:5000. If care were not exercised and if tension and temperature were thus ignored, the accuracy of the measurement would be in the neighbourhood of 1:2500. However, using care, proper tension and temperature correction, an accuracy of 1:5000 can be obtained.

Structural layout is taken to the nearest .005m, pipeline layout and ties for "As Constructed" dimensions generally to the nearest 0.01m.

APPENDIX - 16C

LEVELLING

The Level

The instrument used for accurate levelling is known simply as a "level" as distinct from a Carpenter's level, String level or Hand level.

A level consists of a telescope set on an adjustable horizontal plate which in turn is mounted on a detachable tripod so that the instrument is located at head height. The telescope has a bubble tube mounted on it and rotates about a vertical axis. The line of sight within the telescope is provided by cross hairs as described in Appendix 16D.

Adjusting the Level

The instrument is levelled by adjusting the screws which bear on the horizontal plate with further fine adjustment on the telescope tube itself. In order to ensure that the vertical axis is truly vertical, the level must be adjusted in two positions at right angles to each other. In order to check on the accuracy of the bubble tube, the telescope is then rotated 180° and if the bubble is off, it is brought back half way be levelling screws; the instrument is again rotated 90° and the same bubble error maintained by adjusting the levelling screws. Such a bubble tube error may be corrected by adjusting the capstan screws at one end of the bubble tube. However, since this is rather time consuming, most instrument men will simply allow for this error and live with it for several weeks before making the corrections. Level accuracy is not affected provided the error is known and the appropriate correction made to the level before taking a rod reading.

An error like the above is caused by the bubble tube not being at right angles to the vertical axis and is easily detected. If, however, the telescope line of sight is not at right angles to the vertical axis the error is not readily detected during routine levelling. In order to check for this error the instrument man must carry out the "Three Peg Test". This should be carried out at least once every six months and whenever a new instrument is used.

Three Peg Test

Three points, A, B, and C are set in a straight line such that AB=BC=30m. The instrument is set up at B and elevation of A and C are shot and the difference in elevation obtained. The instrument is then moved close to A, (about 3m away), and A and C are again shot. The difference in elevation should be the same as before. If it is not, the telescope is focused on C, the capstan screws holding the line of sight cross hair in the telescope are adjusted and the line of sight is moved until the correct reading is obtained at C. The reading on A is again checked and further adjustment is made where necessary until the difference in elevation between A and C is the same as it was when the instrument was set up at B.

Theory of Levelling

1. Definitions:

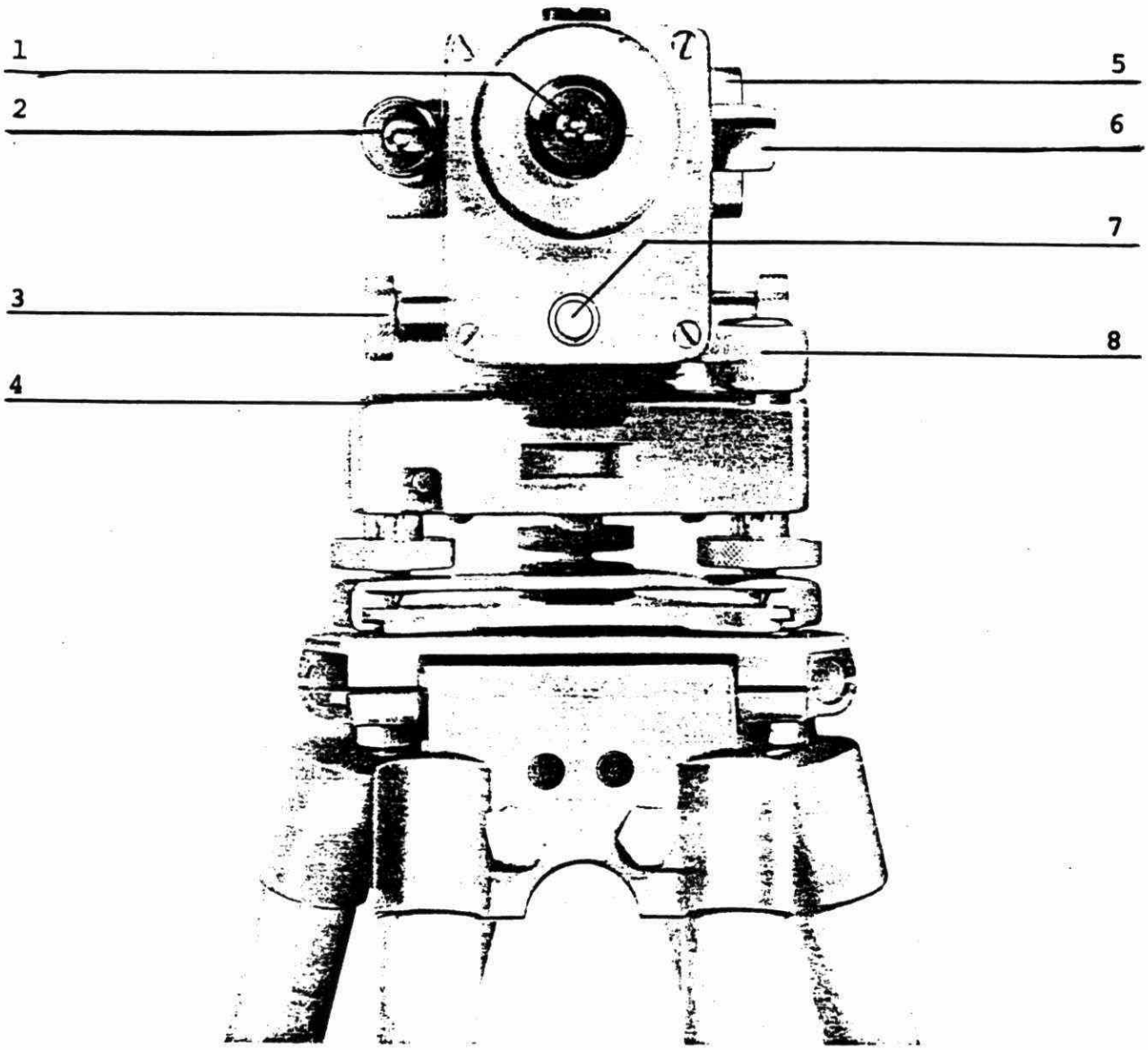
Levelling: is a method used for establishing elevations from a known elevation.

Elevation: elevation of a point is its vertical distance above sea level.

Backsight: (B.S.) is a sight taken with a level to a point of known elevation.

Intermediate Sight: (I.S.) is a sight taken on any point or object to establish its elevation.

Figure 16C-1



Wild NAK2 Universal Automatic Level, with horizontal circle

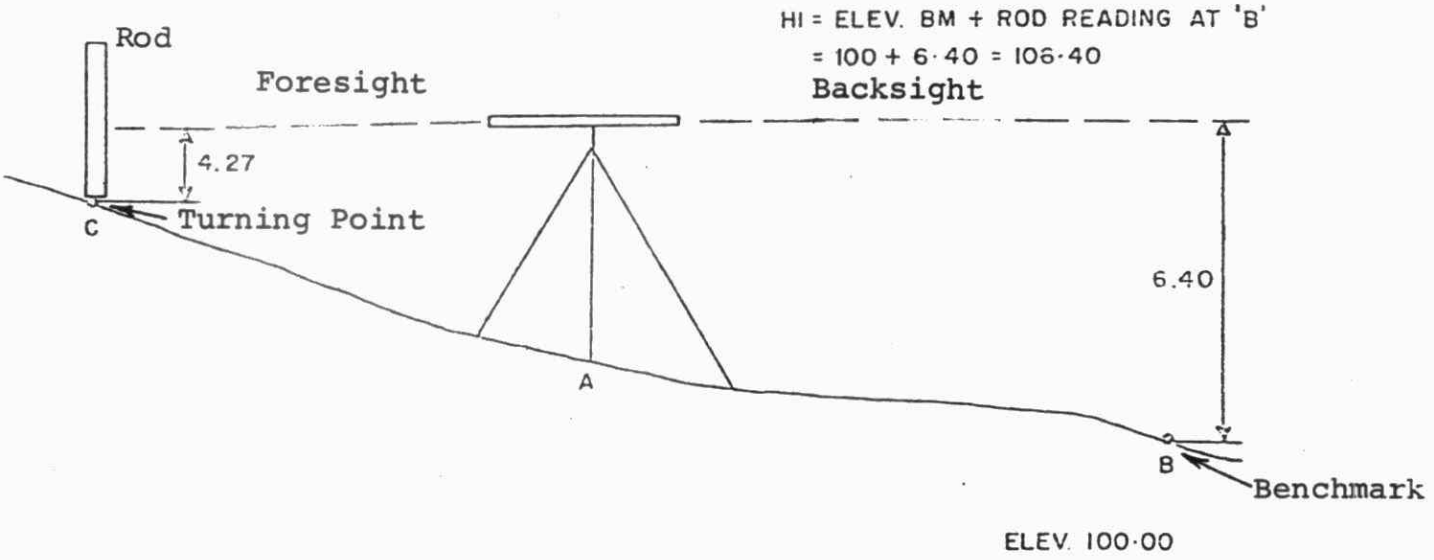
1. Eyepiece
2. Reading microscope for circle
3. Endless horizontal drive screw
4. Knurled rim for setting the circle
5. Focussing knob
6. Mirror for observing the circular bubble
7. Push button
8. Circular level

WILD AUTOMATIC LEVEL

2. Example:

Level is set at point A - elevation unknown. Rod is held on point B - known elevation 100.00. Rod reading as taken from point A is 6.40. Therefore, the line of sight is 6.40 above elevation 100.00, i.e. the height of instrument is 106.40. Now that the H.I. has been established, the elevation of a point C of unknown elevation can be determined by placing the rod on the point 'C' and subtracting the rod reading from the H.I., i.e. elevation of C = 106.40 - 4.27 = 102.13

ELEV 'C' = HI - ROD READING AT 'C'
= 106·40 - 4.27 = 102·13



SAMPLE FIELD NOTES
IF POINT A IS AT STA 1+25
AND POINT B IS AT STA 1+75
POINT C IS AT STA 0+75

(Elevation of B.M. is related to geodetic datum)

STA	B. S	HI	F S	ELEV	REMARKS
1+75	6.40	106.40		100.00	BENCH MARK
0+75			4.27	102.13	T. P

3. Suggestions for the Levelman:

a) Choose a good solid place to set up the level where backsights and the foresights will be about equidistant from the instrument.

b) Avoid sights of more than 60m.

c) Any points of known elevation should be used as turning points as a check on the levelling accuracy to that point.

d) On steep slopes, resist the temptation to take unequal sights. Keep foresights and the backsights equal.

e) Be sure that the level is set correctly before sighting; all four screws in a four-screw instrument should bear firmly and the bubble should be given time to settle before taking reading.

f) It is unnecessary to clamp the spindle and usually it is better not to do so.

g) Keep the bubble out of the direct sunlight - the heat will move it one way or the other, even though the instrument itself may still be level.

4. Suggestions for the Rod Man:

a) Choose suitable turning points. Do not use objects that will settle or move. There should be a definite point for the rod to rest on, e.g. a local high point.

b) Mark all turning points and describe them to the levelman so that they may be easily found again.

c) In choosing a turning point, go as far from the level as the level is from the preceding turning point.

d) Keep the rod plumb, using either a rod level or by waving the rod. One or the other must be used when turning.

Accuracy

For Geodetic Surveying, the accuracy required (in metres) is 0.0032 x No. of kilometres.

For Third order of accuracy (normal levelling) the error in metres must not exceed 0.0095 x No. of kilometres.

i.e. If the distance levelled was four kilometres the allowable error would be:
0.0095 x 4 = 0.038m (38mm)

APPENDIX - 16D

ANGULAR MEASUREMENT BY TRANSIT

The transit is one of the most complex instruments used in construction surveying. The accuracy it produces results from careful manufacture, good design, and the important fact that it can be so operated that it automatically eliminates any residual errors in its construction or its adjustment. The principle upon which this depends is the principle of reversal. It is introduced in the transit by mounting the telescope on the horizontal axis so that it will transit, and thus can be used, both in its normal position called direct and its inverted position called reversed, and by mounting the levels on a part that turns about the vertical axis.

Transits are designed for a number of specific purposes. The type described here is a repeating instrument equipped with a vertical circle and telescope level. This is called an "Engineer's Transit".

The transit consists of three main parts, the alidade at the top, the horizontal circle in the centre, and the levelling head at the base. The alidade includes the entire upper part of the instrument, i.e. the telescope, the standards, the verniers, the plate levels and the inner spindle (inner centre). The horizontal circle includes the outer spindle (repeating centre). The levelling head includes the levelling screws, the bearing for the outer spindle, and the threads that screw on the tripod. The alidade and the horizontal circle can turn independently in a horizontal plane about the vertical axis. The three parts are operated by two clamps, each equipped with a slow motion screw. The upper clamp clamps the circle the alidade and the lower clamp clamps the circle to the levelling head. When a clamp has been tightened, the appropriate slow motion screws can be used to make a fine setting.

For other details of Transit Construction reference should be made to more detailed publications on the subject such as "Plane and Geodetic Surveying" by D. Clark.

Measurement of a Horizontal Angle

By using the upper clamp and the slow motion screw, the zero of the vernier is set precisely opposite the zero on the graduated circle. Then by using the lower clamp and slow motion screw, the telescope is pointed at a target marking the other side of the angle to be measured. The circle remains stationary during this process while the vernier travels around the circle. When the sighting is complete, the position of the vernier gives the value of the angle.

The Telescope Sight

The telescopic sight consists of the following:

1. A pair of cross hairs mounted in a ring or reticle near the rear of the telescope tube.

2. Microscope, or eyepiece, which magnifies the cross hairs and must be focused on them according to the eyesight of the observer.

3. An object glass, or objective lens, which forms an image within the telescope.

The observer observes the cross hairs through the eyepiece and moves the lens backward and forward until the image is clear. This can occur only when the image is on the plane of the cross hairs. Under these conditions, the cross hairs become part of the image, and to the observer, they appear superimposed on the object toward which the telescope is pointed.

If the observer allows himself to look at the image instead of the cross hairs, it is possible to change the focus of the eye so that the image is seen clearly a short distance in front of or behind the cross hairs. The cross hairs will then appear blurred. When the image is not the same distance from the observer as the cross hairs, the cross hairs will move across the image when the eye is moved, just as is the case in observing two objects at different distances with the naked eye. Under these conditions PARALLAX exists and the direction of sight is not sure. When the telescope is properly focused, however, the cross hairs will always appear superimposed on the object, no matter how the eye is moved and therefore the telescopic sight will have a definite direction.

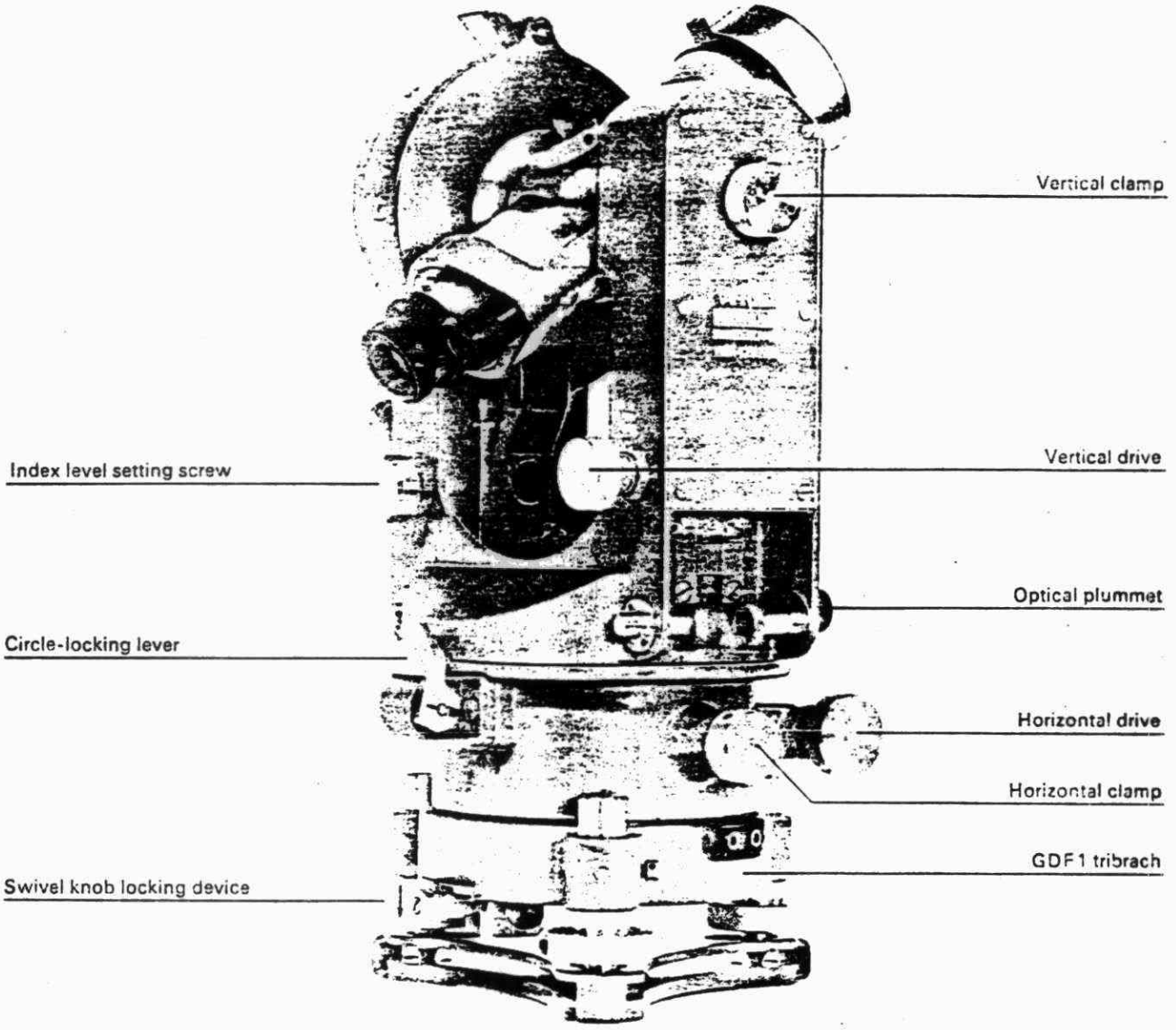
Verniers

A vernier, or vernier scale, is a short auxiliary scale placed along side the graduated scale of an instrument by means of which fractional parts of the least division of the main scale can be measured accurately. The accuracy of the vernier depends on the fact that the eye can more closely determine when two lines coincide than it can estimate the distance between two parallel lines. The graduated circle on most instruments is graduated in half degrees. When this is the case, the vernier is divided into thirty parts covering twenty-nine of the half degree divisions; the readings can be accurately taken to the nearest minute. A single vernier is graduated only to the left of the index mark. A double vernier is graduated both left and right of the index mark. With such verniers, directions can be read clockwise or counterclockwise whenever desired. A clockwise angle is read from right to left and the set of divisions on the vernier to the right of the index mark is used.

Adjustment of the Transit

The inspector should not normally adjust the instrument, however, it is desirable to check the instrument for accuracy by double-centering when running a line, i.e. shooting the angle once with the telescope in the normal position and once after transiting by surveying the telescope 180° vertically and 180° horizontally. Also note any changes in the normal behaviour of the plate level bubbles which may indicate minor damage due to vibration or shock.

Figure 16D-1



WILD ENGINEERS TRANSIT

Care of the Instrument

1. Handle the instrument by the base when not on the tripod. This prevents deflecting the more delicate parts.

2. Never stand the tripod on a smooth surface. The legs may slip outward.

3. Always stand the tripod up carefully. The legs must be wide and firm even when the set-up is not to be used for observations. The wind is not to be used for observations. The wind or a slight touch may knock it over.

4. Never leave the instrument unattended unless precautions have been made for its protection.

5. Never subject the instrument to vibration. Vibration may put the instrument out of adjustment.

6. Never force the instrument. If the telescope or alidade do not turn easily, do not continue to use the instrument. Such usage might damage a bearing.

7. Keep the instrument in its case when it is not in use.

8. Place it in the case so that only the base is in contact with the case. Keep all three transit clamps tight. This reduces chances of vibration.

9. Keep the instruments free from dust and rapid changes in temperature. Dust ruins the finish and the bearings. Because of the nature of Municipal Surveying, it is inevitable that dust will collect on the instrument. When dust appears on the vernier scale, the instrument should be sent out for professional attention. Rapid temperature changes will introduce moisture into the telescope tube causing fog. Sometimes the telescope tube must be dismantled to remove this fog.

10. If the instrument is wet, let it dry be itself. Manual drying ruins the finish and smears the glass and graduations.

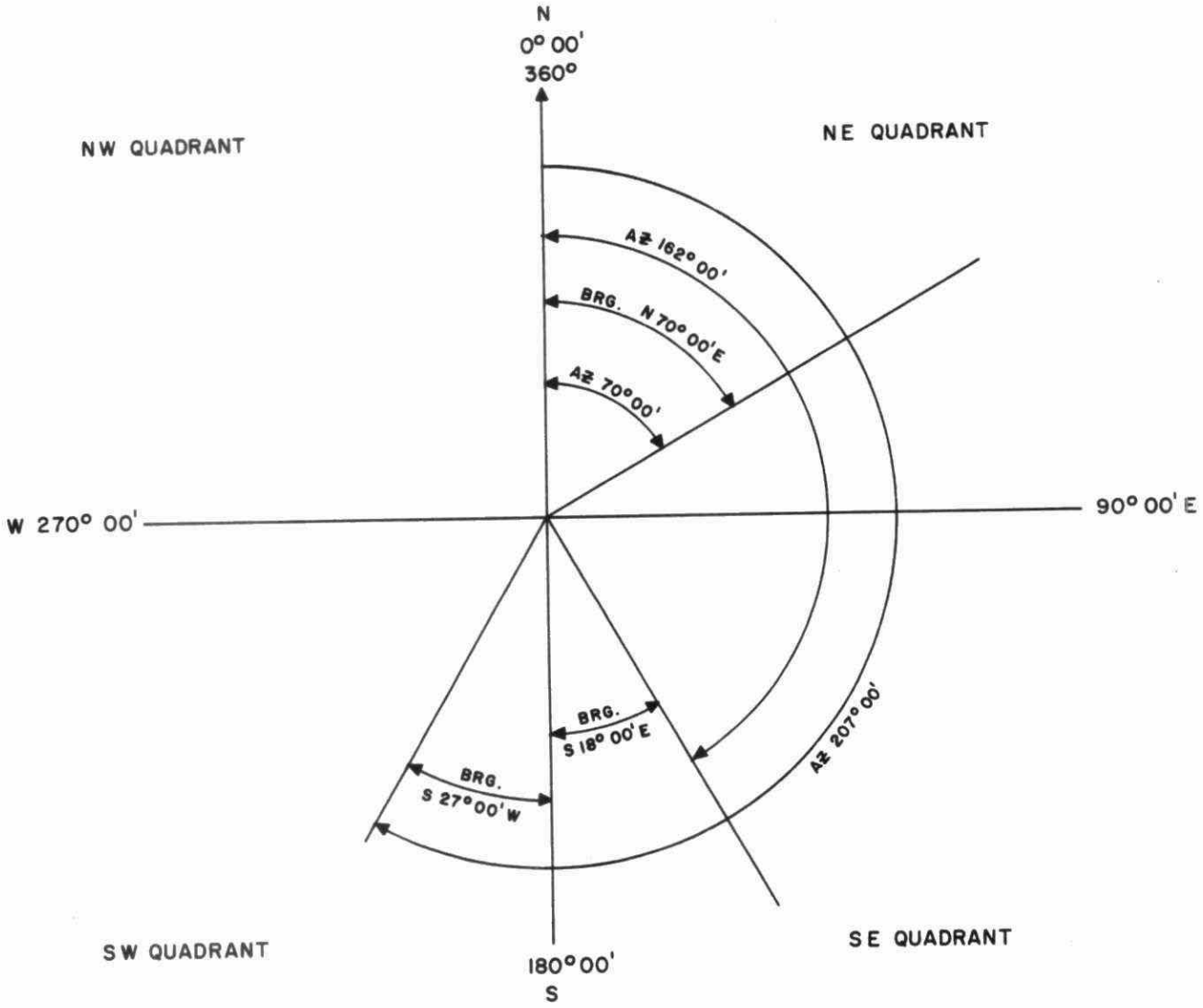
BEARINGS AND AZIMUTHS

The "Azimuth" is the clockwise angle measured from North.

The "Bearing" is the angle measured east and west of north or east or west of south. All readings can therefore by interpreted from a point and fall into four quadrants as follows:

North - East
South - East
South - West
North - West

All survey plans are referred to a bearing and all boundary lines, street lines, etc., have a bearing shown on them.



APPENDIX - 16E

THE USE OF LASERS IN SEWER CONSTRUCTION

Prepared by Spectra-Physics

INTRODUCTION

Sewer lasers have become an accepted method for providing line and grade control for the laying of gravity flow sewer pipe. Today, you will find that almost 50% of all sewer contractors use sewer lasers.

As inspectors, responsible for the satisfactory construction and completion of a sewer job, it is your responsibility to ensure that accurate line and grade is achieved by the contractor. Therefore, you must understand his methods for achieving line and grade control so that you can do a better job of inspection.

TRADITIONAL METHODS FOR PROVIDING LINE & GRADE CONTROL

1. Human Eyeball
   - very inaccurate but fast; unacceptable for laying sewer pipe in almost all cases.

2. Carpenter's Level
   - achieve grade control by placing level on each joint of pipe and knowing how much the bubble should be off
   - certainly not very accurate
   - unacceptable for anything but a rough check

3. Batter boards (2 systems, both referred to as batter boards)

a) those that span across a trench at regular intervals with nails in the top to give centerline

- measure down, usually a number of whole metres, to set invert of pipe at correct elevation
- fairly accurate, acceptable for most underground construction
- but awkward and time-consuming
- not used very much any more

b) "boning arms"

- most common of traditional methods
- stakes at regular intervals along side of trench with cross-pieces set at a given distance above desired invert elevation
- foreman sights along cross-pieces at a "story pole" that has a cross-piece on it, the same given distance above its toe which is resting on the invert of the pipe
- line is usually obtained by measuring over from offset hubs and dropping a plumb bob
- accuracy is generally acceptable, however accuracy attained depends very much on individual doing the sighting.

4. String Lines

- using the offset hubs, string line is set along side of trench at a certain offset from desired centerline of pipe and a given distance above desired invert elevation
- line is obtained by measuring over from string line and dropping a plumb bob
- grade is obtained by hanging a string between the story pole and the string line and checking its levelness with a little level vial hooked to the string
- accuracy can vary and it does take time.

5. Survey Instruments

a) Engineers Level

- generally used to check grade only by setting up at side of trench and "shooting" every pipe, by taking readings on rod held on pipe
- generally used only where grade is critical or to check other methods.

b) Engineers Transit

- can be set up behind manhole on line and then on grade by tilting the telescope, so that you can take the same reading on the rod each time
- requires an instrument man to take shots
- very accurate, but very expensive having an instrument man there all the time.

SUMMARY

1. These techniques have remained essentially the same for 200 years.

2. Accuracy attained is generally considered acceptable but it is solely dependent upon the individuals involved. There can be wide discrepancies between crews or between the contractor and the inspector.

3. Usually 2 men are required to check line and grade.

4. These techniques all require a skilled person, usually the foreman, to check line and grade. This takes the foreman away from other important supervisory tasks.

5. Typically, these techniques are time-consuming as well as being cumbersome.

6. Generally, increased productivity can only be attained at the expense of decreased accuracy. This causes the contractor and the inspector to be in conflict.

The advent and acceptance of sewer lasers is a direct response to the disadvantages of these traditional methods. Most progressive sewer contractors now use sewer lasers to provide line and grade control.

THE USE OF LASERS FOR PROVIDING LINE AND GRADE CONTROL

1. What is a LASER?
   - acronym for L.ight A.mplification by S.timulated E.mission of R.adiation
   - invented in late 1950's
   - can make anything "lase"; carbon dioxide, ruby, etc.
   - laser we use is a low-power gas laser
   - a sealed tube filled with mixture of helium and neon
   - voltage drop created in tube
   - excites molecules which then collide and emit photons of light
   - a 90% mirror at one end and a 100% mirror at other end
   - light emitted is monochromatic light, all of the same wave length and in phase
   - therefore a perfectly straight line of light than can be used for alignment purposes
   - perfectly safe - will not harm you - simply red light, nevertheless, should not look directly into beam.

2. What is a sewer laser?
   - laser tube packaged in a rugged, waterproof casing with accessories to allow laser beam to be set up coincident with theoretical centerline of pipe
   - runs off of 12-volt battery
   - today, most sewer lasers have features and accessories designed to provide reliability and to facilitate fast, accurate set-ups.

3. Basic procedure for using a sewer laser:
   a) set up sewer laser at beginning of run of pipe
      - dial the Grade to set the laser beam on the desired per cent of grade
      - set the laser beam at the correct height above the desired invert elevation
      - level the laser or allow it to self-level
      - set the laser beam on the desired centerline of the pipe

b) generally, the laser beam is set up to shoot through the pipe although it can be used over the top of the pipe

c) using special transparent plastic targets set in the bell end of the pipe, the pipe is centered on the laser beam; it is then both on line and on grade

d) can also use the laser beam as reference for keeping the back-hoe on line, checking mud-grade, and checking the bedding.

4. Evolution of Sewer Lasers

- since their introduction in the late 1960's, sewer lasers have gone through several stages of evolution.

1st Generation Sewer Lasers

- such as Spectra-Physic's Model 655 Dialgrade
- analogous to the Model T in cars
- simple, functional, a solution to a problem
- many are still being used today

2nd Generation Sewer Lasers

- such as Spectra-Physic's Model 855 Dialgrade
- introduced in early 1970's
- refinements of earlier models allowing the sewer laser to be set up faster and easier.

3rd Generation Sewer Lasers" Self-Levelling

- such as Spectra-Physic's Model 955 Dialgrade SL
- introduced in early 1974 and incorporated a new concept: electronic self-levelling
- development of self-levelling was a response to a need
- to be able to leave a laser unattended and have confidence in its remaining level
- to know when the laser is moved off grade
- to further reduce time required for set-ups and to save time previously required for checking and adjusting laser occasionally.

5. The Concept of Self-Levelling

a) "automatic" engineers levels are self-levelling
   - uses an optical compensator - a small lens or mirror-like prism that hangs in pendulum-fashion on hair thin wires or is suspended on miniature ball bearings
   - much faster to set up, just rough-level the instrument, the compensator takes care of the rest

b) self-levelling in a laser is even more important
   - you cannot constantly watch it like an instrument to check its level, you must have confidence in it remaining level
   - if it does get knocked off level, there should be a system to warn you when it is not level
   - any self-levelling system in a sewer laser must be fairly rugged because it gets treated like a tool, not an instrument.

c) to develop a reliable self-levelling system for lasers, Spectra-Physics reviewed the known self-levelling methods:
   i) optical compensator
      - fine for instruments but too fragile for lasers
      - very narrow self-levelling range, +- ¼°
      - cross-axis rotation reduces accuracy of self-levelling device, due to side friction
      - subject to vibration, difficult to damp
   ii) liquid compensator
      - difficult to maintain calibration through wide changes in temperature.

After reviewing the needs of the sewer contractor, the requirements for self-levelling in lasers, and the known methods, Spectra-Physics developed a new system - Electronic Self-Levelling.

d) Electronic Self-Levelling

- incorporates solid state electronics and the simplest of all known level indicators, the bubble in a level vial, both of which can absorb a lot of hard knocks
- system is designed to constantly sense the position of the level vial and uses the resulting signal to control an electronic actuator that keeps the laser precisely level at all times.

- system is designed to constantly sense the position of the level vial and uses the resulting signal to control an electronic actuator that keeps the laser precisely level at all times.

What Electronic Self-Levelling means for a Sewer Laser:

i) a much more rugged, reliable sewer laser
- can take the knocks that it will get on sewer job

ii) a much larger self-levelling range of +-1°
- faster to set up
- manual corrections do not have to be made as often

iii) no cross-axis sensitivity
- grade accuracy not affected by rotation of laser

iv) vibration free laser beam
- due to inherent damping of electronics

v) not affected by temperature changes
- grade accuracy maintained

vi) Early Warning System can be provided
- laser beam flashes on and off when laser is not level
- so crew will not lay pipe to a bad beam.

APPLICATIONS

Sewer lasers can be used on any job under just about any job-site condition. Different support accessories are generally provided to allow this flexibility. While every sewer job is different, if you know the basic procedures and the most common set-ups, you should be able to solve any situation you encounter.

Basic Procedure - Just remember the four main steps:

1. Dial in the correct per cent of Grade.
2. Level the laser or allow it to self-level.
3. Set the laser beam at the right height.
4. Set the laser beam on the correct line.

1. and 2. are self-explanatory and straight forward.

3. Set the laser beam at the right height.

There are four basic methods recommended to set the Spectra-Physics Model 955 Dialgrade SL laser beam at the correct height above invert.

a) Off the manhole floor: If you are starting out from a precast manhole, already set in place, sometimes you can simply measure off the manhole floor. If the manhole was set at the right depth, and the thickness of the manhole was set at the right depth, and the thickness of the manhole floor is known, you can work back to the correct laser beam height above invert very easily by holding a wood rule in front of the laser beam. Just set the laser spot at the right distance off the manhole floor. The Dialgrade SL must be set at the correct per cent of grade before using this method.

b) 966 Hi-Line Calibrated Rods: When using the 966 Hi-Line scope-on-a-pole accessory with your Dialgrade SL, the Hi-Line sighting scope can be rotated to sight a Philadelphia Rod resting on an offset hub. Remember the Hi-Line sighting scope can be rotated 360° on the rod. To set the laser beam height above invert, level the 966 sighting scope and determine HI above the offset hub by taking a reading off the Philadelphia Rod. Add the HI above the hub, to distance from hub to invert called out on the cut sheet. Then subtract the radius of the pipe I.D. The answer is the distance from the centerline of the Hi-Line sighting scope to the center of the Dialgrade SL laser beam. Using the graduations on the Hi-Line calibrated rods set the laser beam at the correct distance from the center of the sighting scope by using the reference lines and edges that indicate the center of the laser beam on the Dialgrade SL laser head.

c) Philadelphia Rod and Engineers Level or Transit: First set up the Dialgrade SL, over the start point, as previously described. Then determine HI be setting a Philadelphia Rod on the manhole center offset hub and taking a reading. Add the HI and the distance from the hub to invert. The answer is the distance from the centerline of the surveying instrument telescope to the invert of the pipe. From this answer subtract the radius of the pipe I.D. and 174mm (the distance from the top of the Dialgrade SL mounting bracket to the center of the laser beam). The answer is the correct distance from the centerline of the surveying instrument telescope to the top of the Dialgrade SL mounting bracket. Now set the Philadelphia Rod on the reference mark on top of the Dialgrade SL mounting bracket. Take a reading. Compare this reading with the previous step and adjust the Dialgrade SL up or down as required using the calibrated vertical pole.

d) Taping from a Batter Board: If an engineer's level or transit is not handy, depth to invert can be obtained easily be installing a batter board over the center of the manhole. Measure down from the batter board edge to the top of the Dialgrade SL mounting bracket with a plumbed tape measure. The correct distance is the distance from the batter board edge to invert less the radius of the pipe I.D., less 174mm from the center of the laser beam to the top of the Dialgrade SL mounting bracket.

4. Set the Laser Beam on the Correct Line

Two points are needed to define a line. For sewer lasers, the two points are generally determined by referring to the offset hubs previously placed. These may be the offset hubs for the manhole centers or even the offset hubs between manholes set to maintain line.

There are three basic methods recommended to set the Dialgrade SL laser beam on line.

a) Dropping plumb lines, measured over from offset hubs.

This method is similar to the methods used to set the centreline of pipe, when using batter boards and string lines. First, determine the start point, or the position of the laser, by measuring over from the offset hub the offset distance and drop a plumb bob. The tape or pole used to measure over should be level and at 90° to the flowline to give a correct measurement.

A second point, at least 6m away from the first, is then determined using the same method. Then the laser beam is aimed at the second point using the tangent adjustment. Usually, 6m or so of trench must be excavated first in order to set the second point.

Line should then be checked when 30m or so of pipe has been laid so that any final adjustments can be made to the line of the laser beam.

b) Engineers Transit

An engineers transit can be "bucked in" on the desired centerline of the pipe behind the start point of the run of pipe. Then, sighting through the transit, the instrument man can first help locate the laser over the start point and then give a second point at which the laser beam can be aimed. Sometimes an engineers transit can be set up directly over the start point using an accessory known as a "manhole mount". Then it is merely a matter of determining the second point with transit. This method is explained in greater detail below in the section on different set-ups.

c) Using the Spectra-Physics 966 Hi-Line Accessory

This accessory system is designed to allow the telescope, used for giving line, to be mounted on the same axis as the laser. Therefore, when the telescope is adjusted to be on line, the laser is automatically put on line. This system is explained in greater detail below in the section on different set-ups.

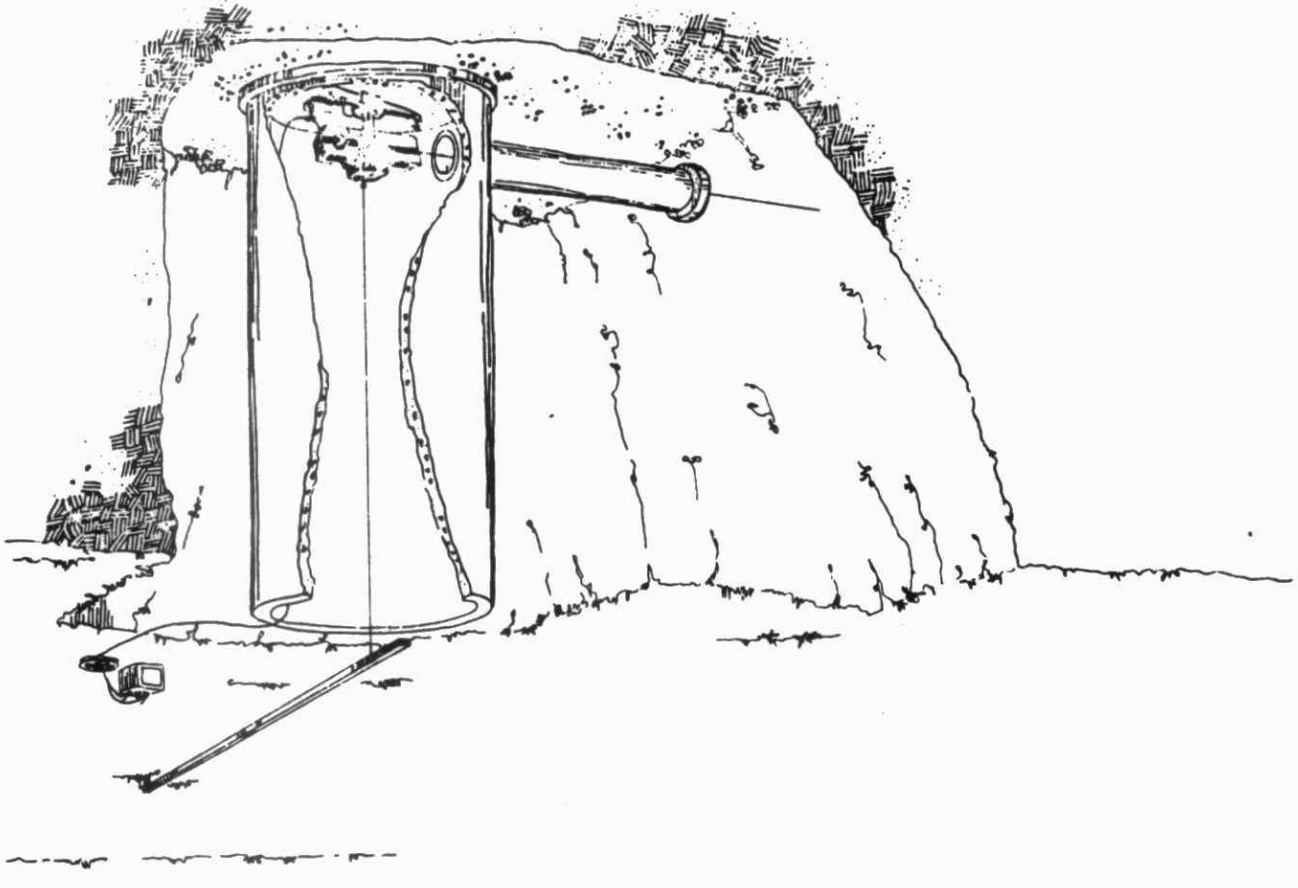
Most Common Set-Ups

1. Trivet Plate
   - the most commonly used set-up by far
   - simplest and quickest to set up
   - can be used in precast manholes with solid bases, large diameter pipes and open excavation with proper support.

MODEL 932 TRIVET PLATE ON A MANHOLE FLOOR

The 932 trivet plate is the fastest way to set up the Dialgrade SL in a precast manhole. The trivet plate has a three screw level system and a calibrated pole with a bullseye level on the top. You should think of it as a baseplate and vertical pole all in one piece. A line adjustment knob is built into the trivet plate that rotates the pole to put the Dialgrade SL laser beam on line.

Figure 16E-1



The 932 is set on the flowline the same way the baseplate is. When the Dialgrade SL is clamped to the trivet plate pole the center of the laser beam is automatically over the rotation point of the pole.

The trivet plate is calibrated from the top down. So when you set a Philadelphia Rod on top of the pole the graduations read downward off the end of the Philadelphia Rod. This makes it easier to determine laser beam height above invert. The Philadelphia Rod can also be set on top of the Dialgrade SL mounting bracket as described in Using the Dialgrade SL section.

Determine line with a stringline and plumb bob, transit on a 607 manhole mount, etc., as described in Using the Dialgrade SL and Major Accessories section. "Dial in" per cent of grade and go to work laying pipe.

The weight of the trivet plate and Dialgrade SL combination plus the plumb bob points on the levelling feet make this setup very stable.

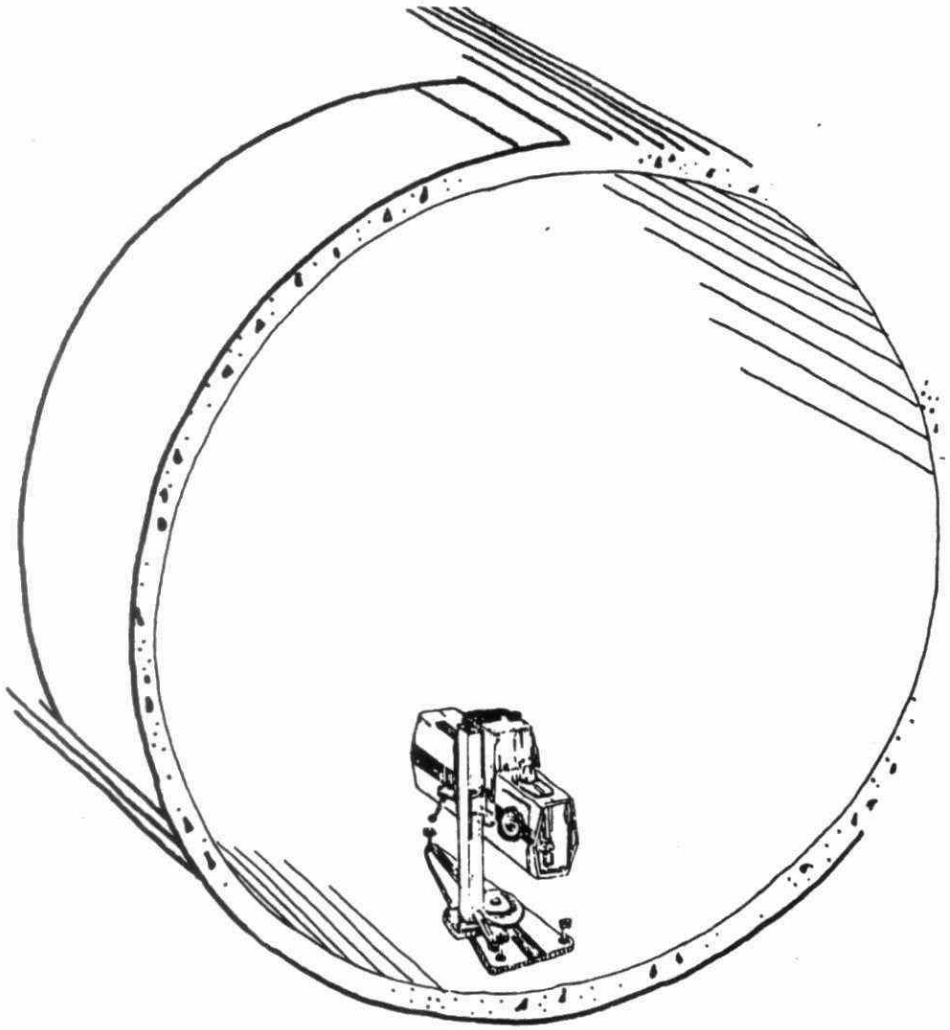
MODEL 932 TRIVET PLATE FOR LARGE PIPE

The 932 is the fastest way to set up in large pipe. The trivet plate simple rests in the invert of the pipe like it does on a manhole floor.

Set up is the same as the procedure for a manhole floor. However, there is a trick or two that makes large pipe set ups with the 932 trivet plate a snap.

The 932 can also be set on cement blocks in front of the first joint if you prefer to set the first joint of pipe with the laser beam. Finding flowline and determining laser beam height above invert is done as described in Using the Dialgrade SL and Major Accessories section.

Figure 16E-2



2. Vertical Poles and Cross-Arm Braces
   - provides a much more stable set-up and can be used in drop manholes or poured invert manholes, manholes with solid bases, large diameter pipe and can be combined with the 966 Hi-Line Accessory to provide a simple way to determine line.

MANHOLE WITH POURED INVERT OR RAISED INVERT

If a concreted round invert has been poured in your manhole floor or if a raised invert is required you can still use the Dialgrade SL and get a perfect job.

Hang a stringline and plumb bob down the center of the manhole opening. This provides a centerline reference that makes it easier to line up crossarm braces, clamp adjusters and the vertical pole before making final adjustments.

Set the crossarm braces in place so that they just touch the stringline and are parallel when looking down on them from above. They should be set about 1m one above the other. (However, they can be set farther apart when using 966 Hi-Line.)

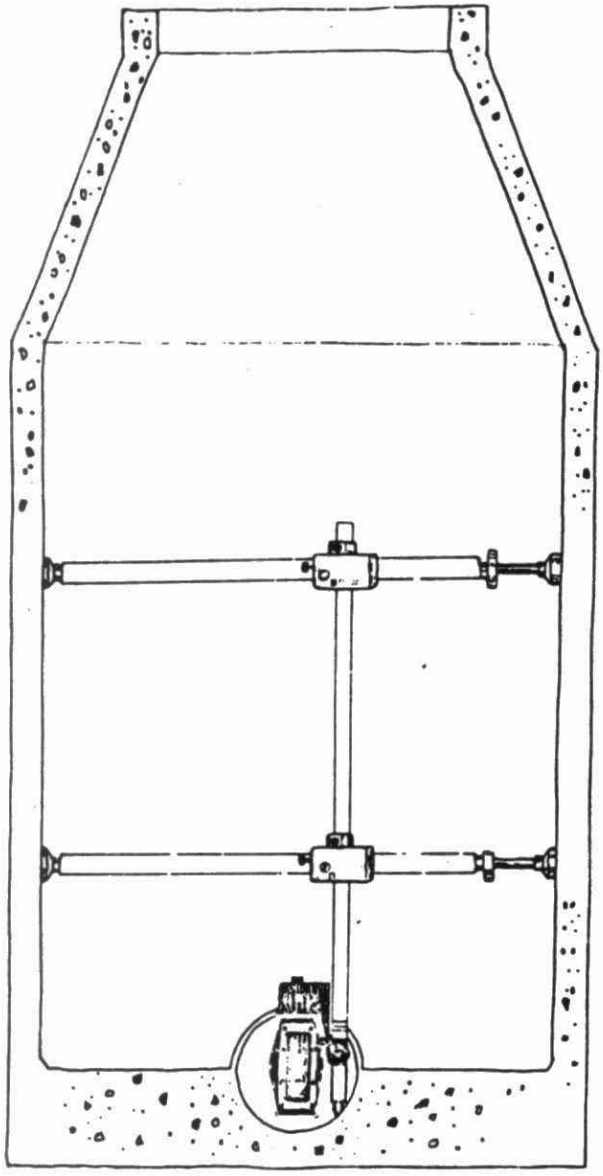
Assemble a Hi-Line clamp adjuster to each crossarm brace. Align the Hi-Line clamp adjusters on the crossarms so that you can look down through the square vertical pole holes to an imaginary point 63mm to the left of the flowline of the poured invert, when facing the direction the pipe will be layed.

Assemble a vertical pole or stack of Hi-Line rods through the Hi-Line clamp adjuster.

If the set up is for a poured invert the vertical poles should be pushed down against the side of the invert with the steel point on the bottom rod touching the concrete.

If the dual crossarm brace set up is for a raised manhole invert the crossarms should be positioned so that the laser beam height above the invert falls between the crossarms so that the Dialgrade SL can be clamped on between them.

Figure 16E-3



If the 966 Hi-Line Accessory is used with two crossarm braces the bottom Hi-Line clamp adjuster must freely rotate when adjustments for line are made with the top Hi-Line clamp adjuster.

Now the vertical pole or Hi-Line rods must be plumbed and adjusted so that the Dialgrade SL laser beam will be on flowline. This is usually done by simply measuring the center of the poured invert as cast in the floor and setting up the laser beam to that point on the flowline. Now get another point on the flowline up ahead and adjust the laser beam to split it. See Using The Dialgrade SL section.

With a raised manhole invert, start point is in space. Consequently, a stringline and plumb bob must be dropped that is centered on flowline. The vertical pole hanging is space must be adjusted so that it is plumb and parallel to the stringline and 60mm on the left side of it facing the direction the pipe will be layed. When the Dialgrade SL is clamped to the vertical pole the laser beam will be on the flowline of the raised invert. Beam height above invert and final line adjustments can now be determined using the methods outlined in Using The Dialgrade SL section.

These set ups are very rigid and give excellent stability. The Dialgrade SL with electronic self-levelling gives you perfect grade insurance in these two tough set up situations.

MANHOLE WITH SOLID BOTTOM

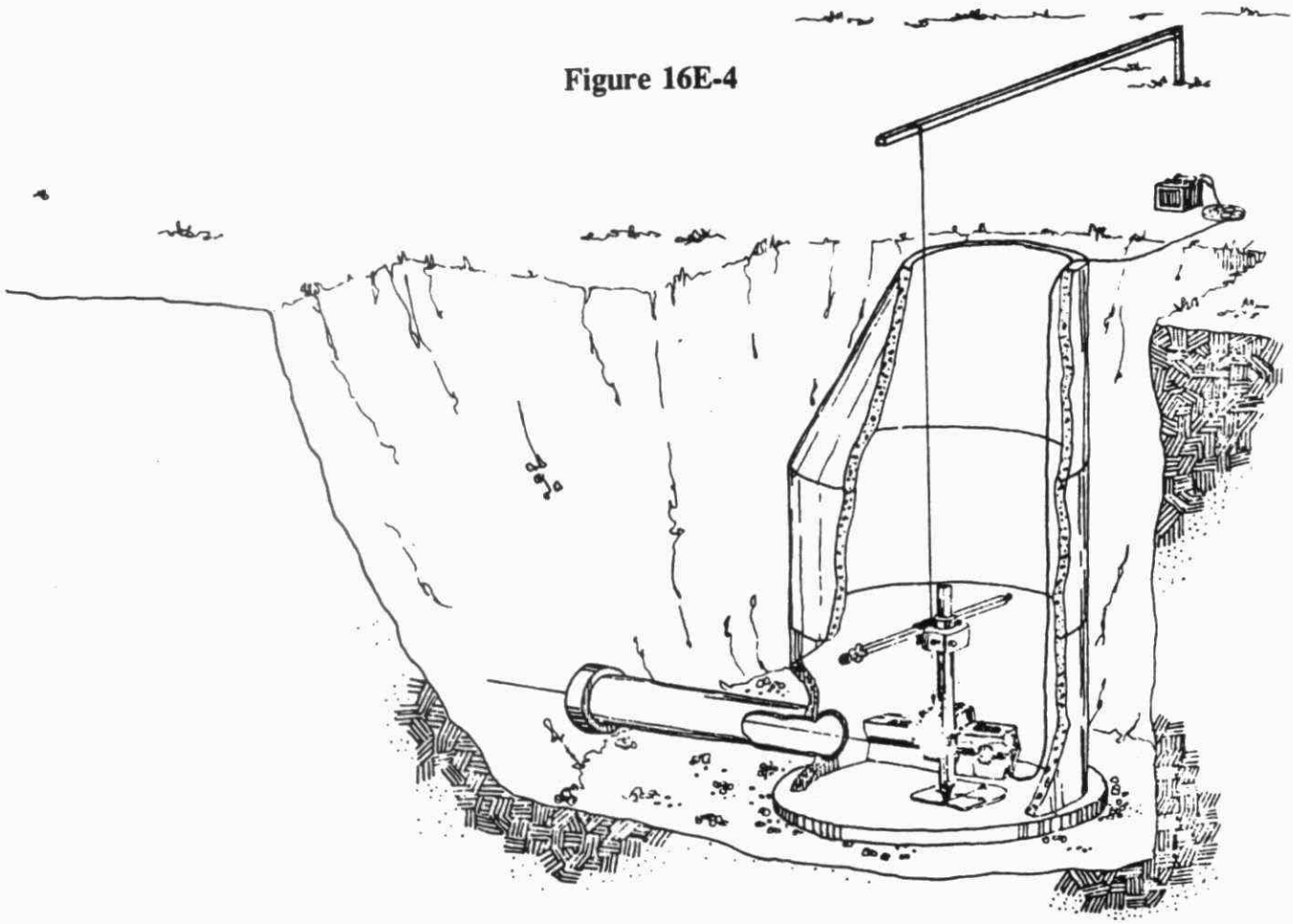
We have already discussed use of the Dialgrade SL with the 966 Hi-Line Accessory in the section on Using Major Accessories. The Dialgrade SL can also be set up in a manhole using only a short vertical pole, one crossarm brace and a baseplate.

1. Start with a point along the center of the pipe. Place the baseplate over the start point.

2. Only one side of the crossarm brace extends. Hold one side against the manhole wall and extend the other end until it contacts the opposite side. The speed nut on the crossarm brace makes it fast and easy.

3. Assemble the Hi-Line clamp adjuster to the crossarm brace. Sight down through the square hole in the clamp adjuster and align it with the hole on the left edge of the baseplate as you look toward the direction the pipe will be layed.

Figure 16E-4



4. Set the vertical pole in the baseplate. Secure the vertical pole to the crossarm brace with the Hi-Line clamp adjuster. Plumb the vertical pole using the bullseye level on top for reference. This is done by moving the crossarm brace and sliding the Hi-Line clamp adjuster back and forth.

5. With the crossarm brace set in place and the Hi-Line clamp adjuster and vertical pole assembled it is a very solid set up.

6. The Hi-Line clamp adjuster provides 360° rotation and +- 10° fine line adjustment.

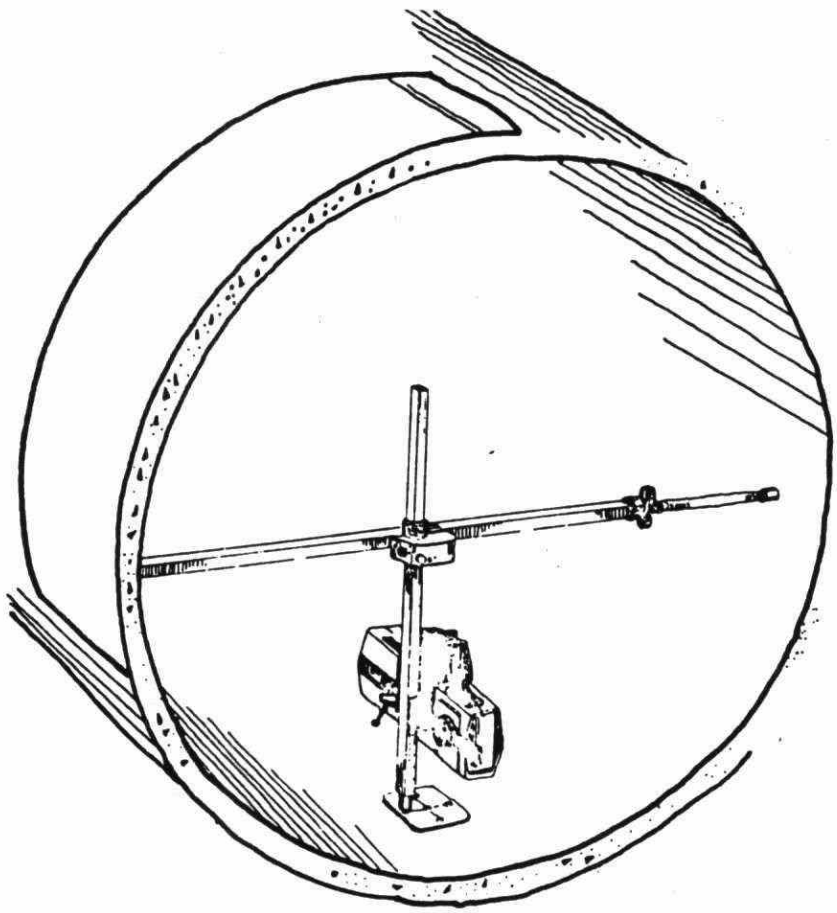
7. Obtain laser beam height above invert and get the laser on line using techniques outlined is Using The 955 Dialgrade SL section.

CROSSARM BRACES IN LARGE PIPE

The same crossarm brace used in a precast manhole setup also works in large pipe. Crossarm braces can be cut as short as 575mm if desired. The standard crossarm brace works in 1200 to 1500mm diameter pipe.

If pipe larger than 1500mm is being layed set the crossarm brace against a 100 x 100 timber put vertically in the pipe.

Figure 16E-5



The first step is to establish a start point in the first joint of pipe. If a transit is available just set the Dialgrade SL baseplate on flowline by lining it up with the vertical crosshair of the transit. If a transit is not available tie a stringline and plumb bob around the center of a small carpenter's level, just like the one you would use for batter boards. Level the carpenter's level on a chord for batter boards. Level the carpenter's level on a chord across the upper ID of the pipe. The plumb bob will hang to the flowline of that joint of pipe. Mark the point and set the baseplate over it. Attach a crossarm brace over the baseplate and install the Hi-Line clamp adjuster sliding it until the square hole is over the proper hole in the baseplate. Install a vertical pole and adjust it plumb. Clamp the Dialgrade SL on the vertical pole. "Dial in" per cent of grade. Set the laser beam to the desired height above invert and get it on flowline using the methods described in Using The Dialgrade SL section.

Take a large pipe target to the other end of the pipe. Level the target in the invert and adjust the bullseye on the target to the same height as the laser beam. Remember the laser beam is already on line and grade. Now your target is set up to lay the remaining pipe.

There are many variations to setting up in large pipe. If you need to keep the pipe floor clear for such things as pipe jacks use two crossarm braces and two Hi-Line clamp adjusters and suspend then laser in the roof of the pipe. Remember the large pipe target can be turned upside down and set on the bell of the pipe.

HI-LINE ACCESSORY SETUP PROCEDURE

The basic setup techniques in the Using Dialgrade SL section remain the same. The Hi-Line Accessory expands the overall utility of the Dialgrade for those contractors who prefer the scope of the pole setup. The manhole and open excavation setup illustrations depict two of the most common applications for the Hi-Line Accessory. The basic alignment steps for both of these set up situations are as follows:

Step No. 1

The Hi-Line Accessary sighting scope can be rotated to sight on an offset hub for determining HI just by unlocking Knob 1. The sighting scope will now rotate 360° horizontally. There is no necessity for unlocking the rod or taking the scope off the rod sitting on the baseplate. Place a Philadelphia Rod on the manhole center hub. Level the Hi-Line sighting scope and take a reading to determine HI. Add the HI to the distance from the centerline of the sighting scope to the invert of the pipe. Then subtract the radius of the pipe I.D. Now you have the desired distance from the centerline. Using the calibrated rods set the Dialgrade SL beam at the determined distance.

Step No. 2

After assembling the Dialgrade as illustrated, making sure the calibrated rods are plumb, it is then necessary to sychrozine the Dialgrade laser head and the Hi-Line Accessory sighting scope. First, excavate a metre or so of trench. Find the laser beam hitting the back of the cut and adjust the Hi-Line sighting scope, Knob 2, until the vertical crosshair splits the laser beam. If the beam is difficult to see in the dirt use a pipe target or shovel to enhance beam visibility in the ditch.

Step No. 3

Next plunge the sighting scope down on the Dialgrade laser head and use the lateral fine adjust knob, Knob 3, to adjust the sighting scope so that the vertical crosshair splits the black line reference mark on the top front end of the Dialgrade.

Step No. 4

Recheck the alignment of the laser beam in the trench by repeating Step No. 2 and make a fine adjustment if necessary. the Dialgrade laser head and the Hi-Line sighting scope are now synchronized in the same vertical plane.

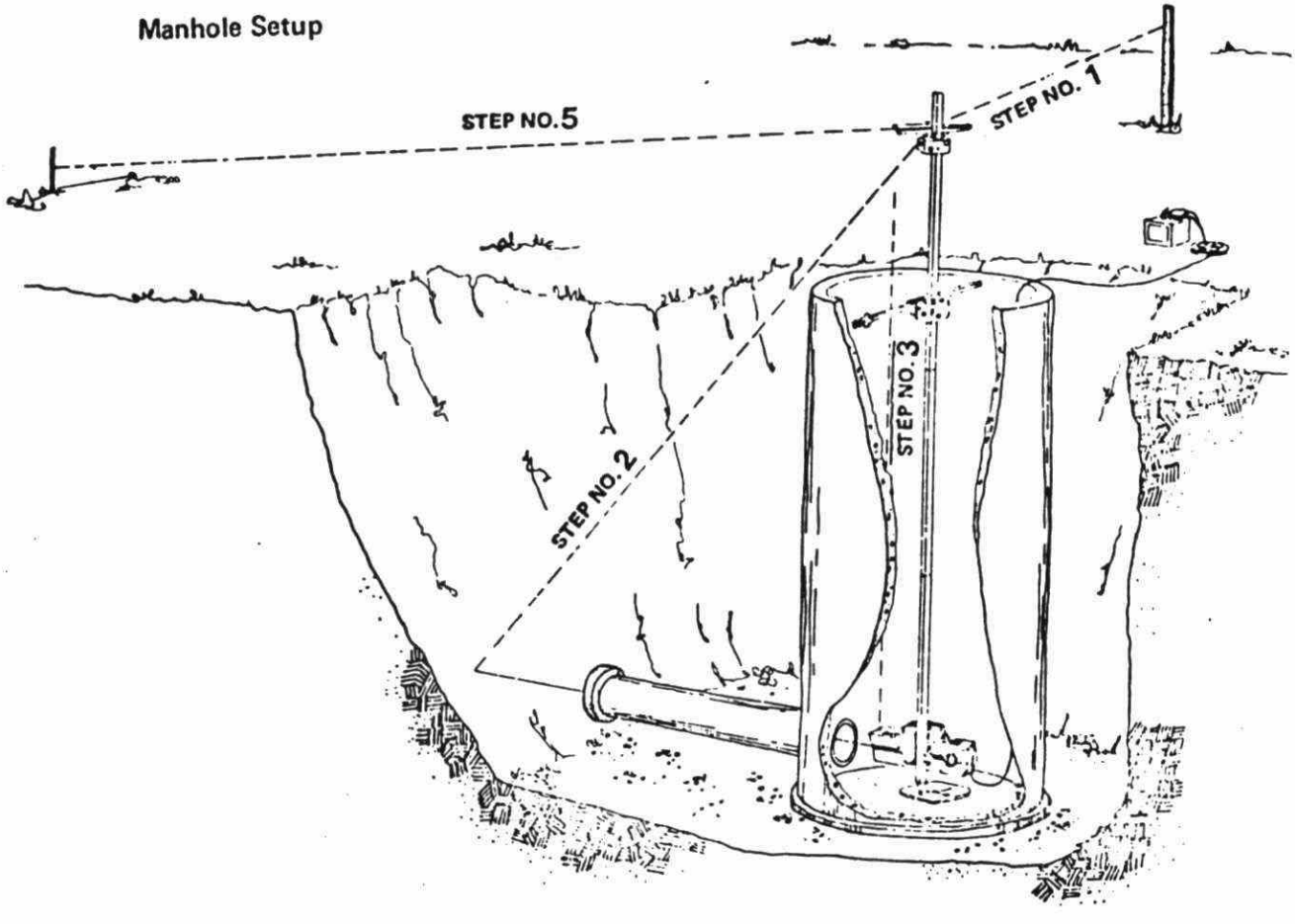
Step No. 5

Sight on the next manhole and align the sighting scope vertical crosshair with the manhole center by using the horizontal fine adjust knob on the Hi-Line clamp adjuster, Knob 2. Since the Laser head is synchronized to the sighting scope the laser beam is now on line.

SIX SIMPLE STEPS TO REMEMBER

1. Determine laser beam height above invert.
2. Plumb the rods.
3. Sight in the trench.
4. Sight down on the laser head reference mark.
5. Resight in the trench.
6. Sight on the next manhole center.

Figure 16E-6



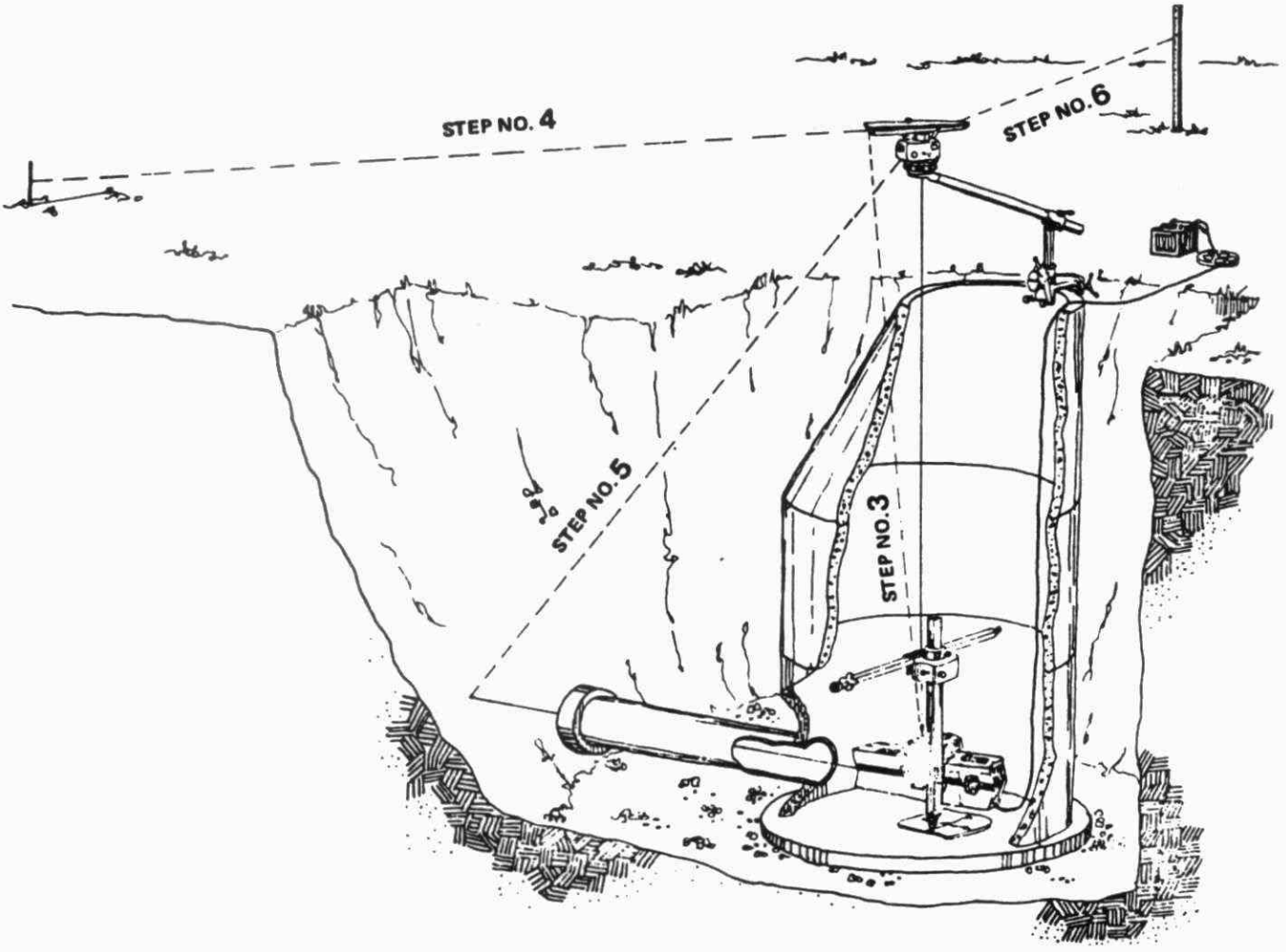
3. Manhole Mount

- the manhole mount accessory can be used to set up either an engineers transit or the 966 Hi-Line Accessory Sighting Scope directly over the start point.

MODEL 607 MANHOLE MOUNT WITH 966 HI-LINE SIGHTING SCOPE & ADAPTOR

In those situations where the complete 966 Hi-Line Accessory cannot be used, such as a very deep manhole, or because of personal preference the Model 868 adaptor can be used that permits use of the 966 Hi-Line sighting scope on the Model 607 manhole mount.

Figure 16E-7



The procedure for using this is to first set up the Dialgrade SL over the start point as previously described. Then adjust the 607 manhole mount adaptor and Hi-Line sighting, so the scope is directly over the reference mark on the top of the Dialgrade SL mounting bracket. This is done by:

Step No. 1

Hang a plumb bob down from the offset wing of the adaptor over the reference mark on top of the Dialgrade SL mounting bracket.

Step No. 2

Plumb the Hi-Line sighting scope adaptor using the bullseye level on top of it. The manhole mount adjustment clamps give all axis freedom. Make sure the plumb bob is still over the reference mark when finished with this step.

Step No. 3

Plumb the vertical crosshair of the Hi-Line sighting scope using the lateral fine adjust knob No. 3. This is done by sighting down the plumb bob string hanging down to the laser or by having someone hold a plumb string on the side of the ditch for you to sight on.

Step No. 4

Sight on the next manhole center and lock up the Hi-Line sighting scope so the vertical crosshair splits it.

Step No. 5

Plunge the Hi-Line sighting scope into the trench approximately 8 metres ahead and adjust the Dialgrade SL laser beam, using the fine adjust knob No. 2 on the Hi-Line clamp adjuster, until the beam splits the vertical crosshair of the scope.

Step No. 6

Find laser beam height above invert as described in using the Dialgrade SL section.

You now have the laser beam on line. With the Hi-Line sighting scope locked on line it can be used as a line check until you arrive at the next manhole. Remember the 607 manhole mount has the same thread so that it can be used with any engineers transit.

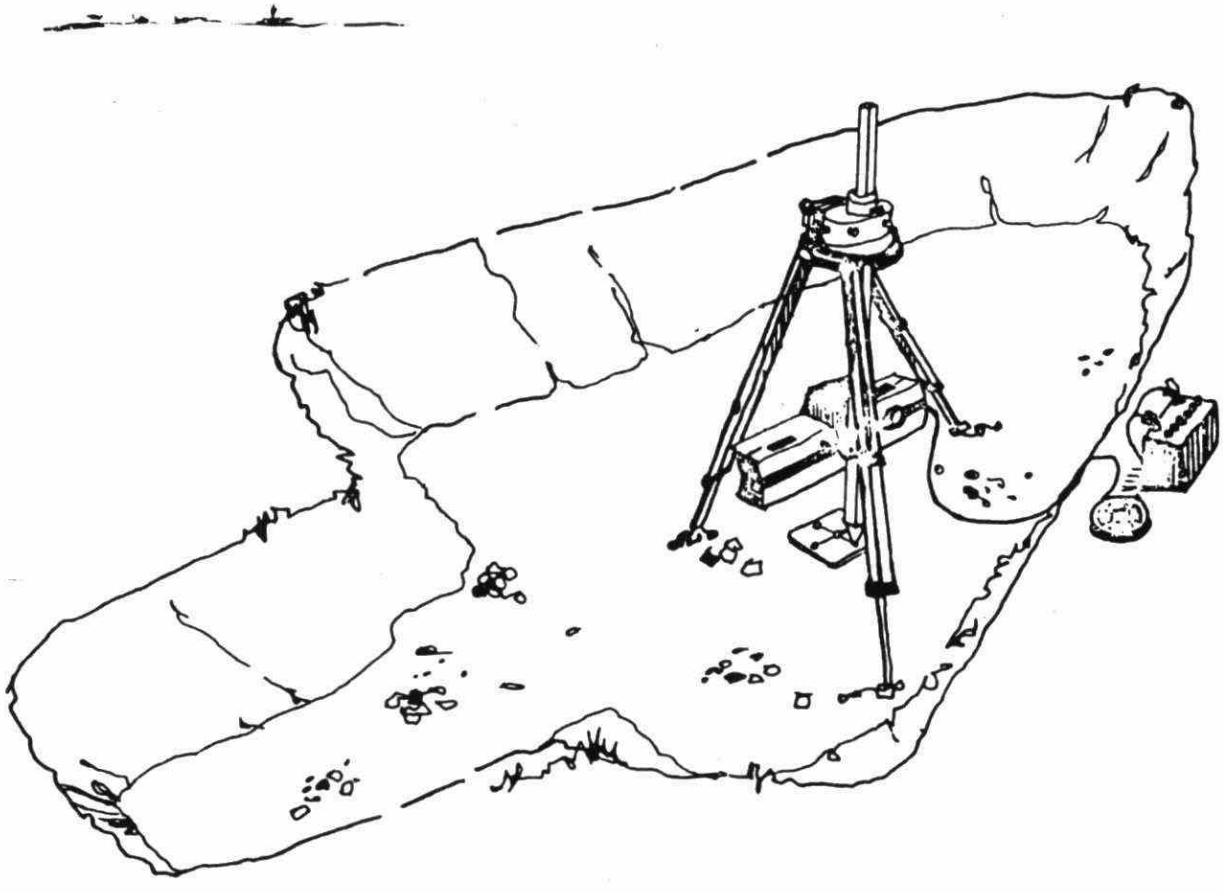
4. Tripod Set-Ups
   - the Dialgrade sewer laser can also be set up in an open excavation using a tripod, a vertical pole and an adaptor
   - the 966 Hi-Line Accessory can also be used with the tripod set-up.

OPEN EXCAVATION WITH A TRIPOD

If your precast manholes didn't arrive on time, you don't need to worry with the Dialgrade SL. Just set up with the Dialgrade SL tripod in the open excavation you have to dig for the manhole anyway.

Now establish a start point on flow line. Place the baseplate over the start point with the center groove lined up parallel to flowline. Step down hard on the baseplate to anchor it in the earth.

Figure 16E-8



Fasten the Hi-Line clamp adjuster to the tripod.

Now set the tripod and clamp adjuster over the hole on the left edge of the baseplate as you face the direction the pipe is being layed. Look down through the square hole in the clamp adjuster to eyeball align it with the baseplate. This usually involves moving the tripod legs around.

Slide the vertical pole through the square hole in the clamp adjuster and put the steel point on the end of the vertical pole in the proper hole in the baseplate.

Now plumb the vertical pole by sliding tripod legs or the clamp adjuster on its bracket. The bullseye level or pole level make this task easy for you.

Now attach the Dialgrade SL to the vertical pole. Find line and beam height above invert as explained in Using The Dialgrade SL section. Your Dialgrade SL is now providing you with electronic self-levelling grade insurance in an open cut on a tripod.

HI-LINE ACCESSORY SETUP PROCEDURE - FOR TRIPOD SET-UP

The basic setup techniques in the Using Dialgrade SL section remain the same. The Hi-Line Accessory expands the overall utility of the Dialgrade for those contractors who prefer the scope on the pole setup. The manhole and open excavation set up illustrations depict two of the most common applications for the Hi-Line Accessory. The basic alignment steps for both of these setup situations are as follows:

Step No. 1

The Hi-Line Accessory sighting scope can be rotated to sight on an offset hub for determining HI just by unlocking Knob 1. The sighting scope will now rotate 360° horizontally. There is no necessity for unlocking the rod or taking the scope off the rod sitting on the baseplate. Place a Philadelphia Rod on the manhole center hub. Level the Hi-Line sighting scope and take a reading to determine HI. Add the HI to the distance from the hub to the invert of the pipe. The answer is distance from the centerline of the sighting scope to the invert of the pipe. Then subtract the radius of the pipe I.D. Now you have the desired distance from the centerline of the sighting scope to the laser beam center. Using the calibrated rods set the Dialgrade SL beam at the determined distance.

Step No. 2

After assembling the Dialgrade as illustrated, making sure the calibrated rods are plumb, it is then necessary to synchronize the Dialgrade laser head and the Hi-Line Accessory sighting scope. First, excavate a few feet of trench. Find the laser beam hitting the back of the cut and adjust the Hi-Line sighting scope, Knob 2, until the vertical crosshair splits the laser beam. If the beam is difficult to see in the dirt use a pipe target or shovel to enhance beam visibility in the ditch.

Step No. 3

Next plunge the sighting scope down on the Dialgrade laser head and use the lateral fine adjust knob, Knob 3, to adjust the sighting scope so that the vertical crosshair splits the black line reference mark on the top front end of the Dialgrade.

Step No. 4

Recheck the alignment of the laser beam in the trench by repeating Step No. 2 and make a fine adjustment if necessary. The Dialgrade laser head and the Hi-Line sighting scope are now synchronized in the same vertical plane.

Step No. 5

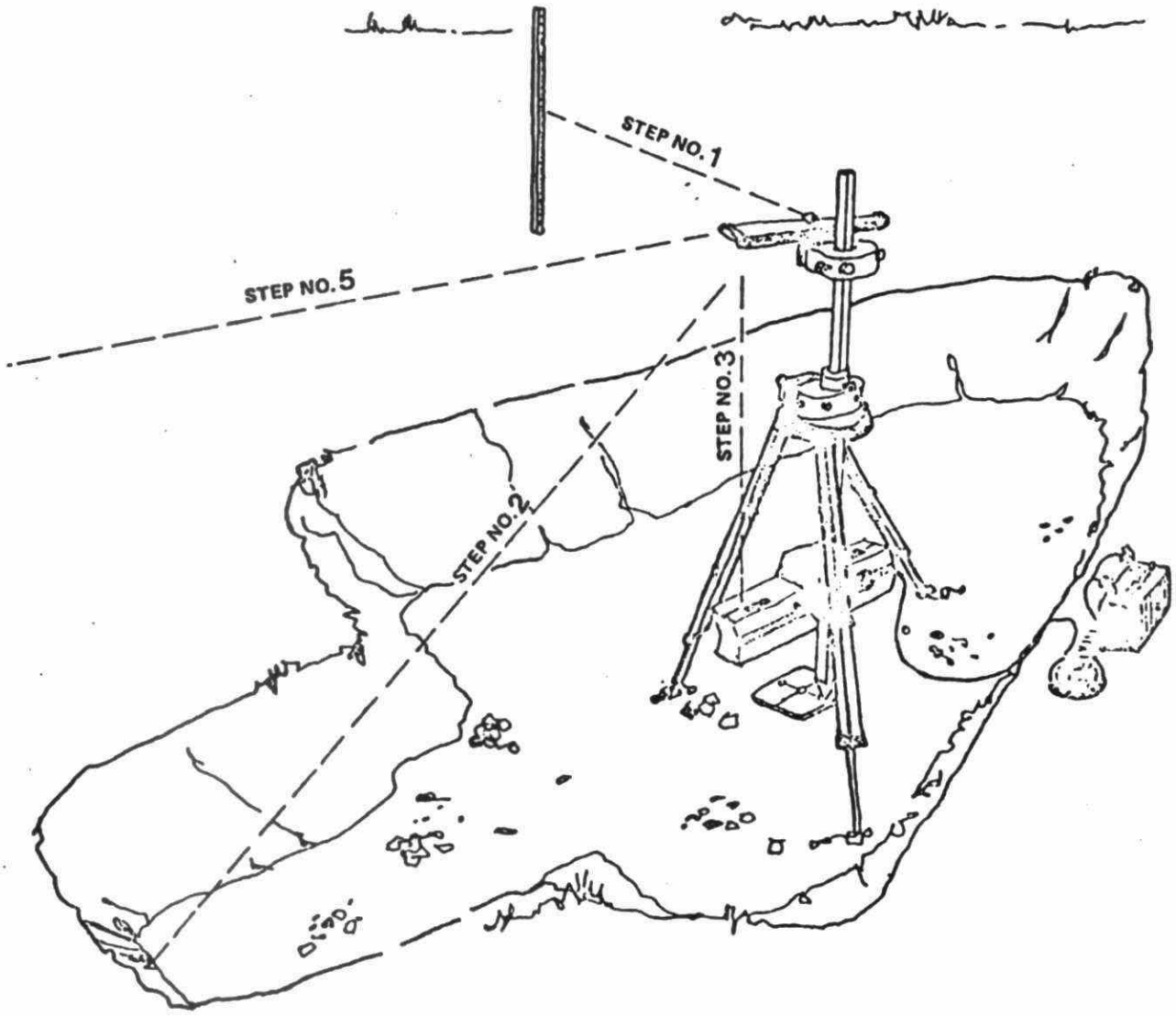
Sight on the next manhole and align the sighting scope vertical crosshair with the manhole center by using the horizontal fine adjust knob on the Hi-Line clamp adjuster, Knob 2. Since the laser head is synchrozined to the sighting scope the laser beam is now on line.

Six Simple Steps to Remember

1. Determine laser beam height above invert.
2. Plumb the rods.
3. Sight in the trench.
4. Sight down on the laser head reference mark.
5. Resight in the trench.
6. Sight on the next manhole center.

Figure 16E-9

Open Excavation Setup



SUBJECT OF SPECIAL INTEREST TO USERS OF SEWER LASERS

Refraction

The Dialgrade SL laser beam is a perfectly straight line of coherent light. Coherent light means it is concentrated in a small diameter and will not diffuse or spread out except at great distance. The fundamental optical principles of reflection and refraction of light also apply to laser light. One of these principles - refraction, can have an effect on the laser beam. Refraction is simply the bending of light at the interface of two substances caused by a difference in the speed of light. Have you ever had the experience of reaching down into water for an object only to find that you grabbed for it in the wrong place? Your aim was fooled by the light refraction between air and the more dense water. The same thing can happen to a laser beam inside a pipe. When air densities in confined places like a pipe line occur due to temperature changes (warm air is less dense than cold air) the laser beam will move usually downward. Air density changes, usually in small pipe, are caused not only by temperature but also by gases and solvents. The vapour from plastic pipe cement, and exhaust gases from a pump or tamper that get temporarily sucked up the pipe cause air density changes.

Refraction of sewer pipe lasers is generally a seasonal thing. The hot summer months is when to look out for it.

The most common cause of laser refraction in a pipeline is installing hot pipe, that has been sitting in the sun at the edge of the trench, into the cool earth at the bottom of the trench. The cool earth immediately begins cooling the underside of the pipe. The air density in the lower half of the pipe changes as the air cools. As the pipeline gets longer the condition gets steadily worse (usually 50 to 60m out). The beam begins to flame on the edges or droop off toward the invert of the pipe. Remember this condition occurs only because of air density changes in the pipe. So look out for hot pipe in a cool ditch, cool pipe left un-backfilled over a cool night followed by a very hot day with sun shining down on the pipe, a pipelayer slopping excessive cement on joints of plastic pipe or a pump exhaust pointing at the pipe opening.

Here are some ways to alleviate these problems:

1. Keep pipe out of the sun if possible.
2. Lay the pipe hot side down.
3. Backfill as soon as possible.
4. Do not slop excessive plastic pipe cement on each joint that gets squeezed into the pipe when a joint is shoved home.

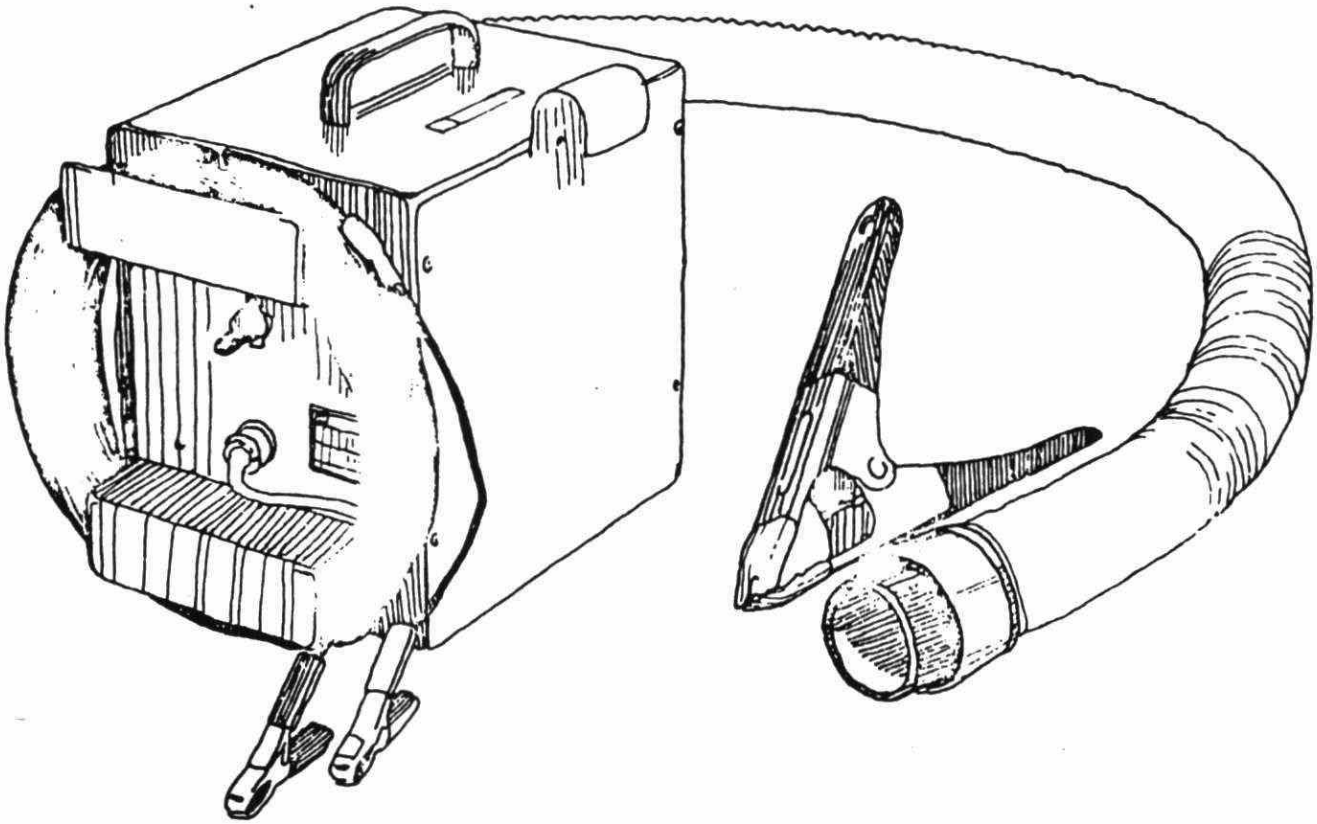
If the above steps do not work use your Blower to re-establish constant air density in the pipe. Sometimes it may take 10 minutes to blow out 80 to 100m of 300mm sanitary sewer pipe.

Refraction usually occurs only in small pipe. However, the condition has been witnessed in sizes up to 1200mm. This is very rare. If refraction occurs in large pipe a larger blower such as a Homelite Model 111B gasoline engine driven unit may be required.

To use Model 809 Blower poke into the end of the pipe and secure with the large alligator clamp provided. Connect the battery clip on the blower power cord to a vehicle battery. Red clip goes to positive battern terminal. Turn on the blower with the on/off switch provided. Fasten the blower to a manhole step with the leather straps to keep it out of the water.

Figure 16E-10

Model 809 Blower



Calibration

It is very easy to check calibration of the Dialgrade SL by double-levelling. The only equipment you need is an engineering rule, the lase head, 932 trivet plate and a battery. Here are the steps:

1. Select a road or field that is fairly level. (It does not have to be truly level, but should not rise more than 100mm in 15 to 30m, or the spot will hit the ground.)

2. Designate two points on the ground approximately 30m apart with a felt tip pen, pencil, or by driving a nail. The distance does not have to be measured. Mark the two points with paint or two tin cans so you can later find them easily.

3. Dial zero grade. Set the laser on the 932 trivet plate or tripod and vertical pole just behind the point lowest in elevation. Connect it to a battery. Aim the beam so it is passing over the two points. (If you do not have a fairly level area, run the test with the laser raised up on a sturdy box at the low end; or mount it on the Vertical Pole supported by the tripod.)

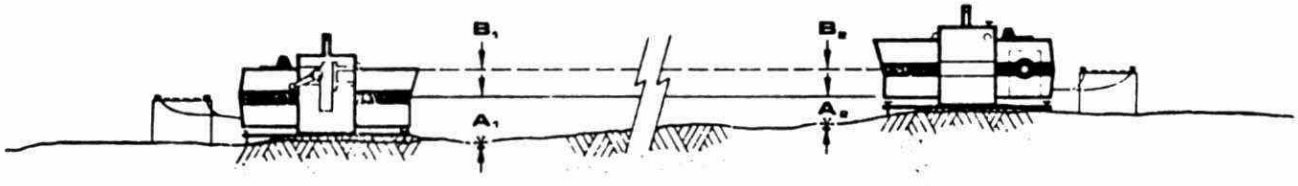
4. Check that the grade indicator is set to zero (level). Allow the Dialgrade SL to self-level.

5. Measure the distance from each point on the ground to the beam center. These distances are shown as A_1 and B_2.

6. Move the laser behind the second spot and repeat the two measurements. These are B_2 and B_1.

7. If the level vial is accurate, then the beam will be truly level with respect to the earth. Hence, the two beams will be exactly parallel to each other at both test positions. Subtracting A_1 and B_1 should give you a number the same as A_2 and B_2 (to within 3mm). If you were working on a truly level road, all readings would be the same, and the two differences would equal zero.

Figure 16E-11



SAMPLE SPECIFICATIONS FOR LASER CONTROL ON SEWER CONSTRUCTION

Line and Grade Control

The engineer shall lay out the job by providing an offset hub at each manhole location. The engineer shall provide cut sheets for the contractor's use in executing the work. The cut sheets shall show the amount of cut from hub elevation to pipe invert elevation at each hub. Also, the cut sheet shall show the amount of grade in per cent for each section of pipeline.

In executing the work, the contractor shall utilize a laser instrument in conjunction with the engineer's layout work. The grade instrument used shall have the grade indicator dial graduated to read directly to 0.01% grade with inter-polation possible to 0.001% grade. The instrument shall be a factory-built, quality-controlled, precision instrument, capable of controlling grade to a tolerance of plus or minus 0.01% grade through a grade range of 10% to +20%.

The laser for the grade instrument shall be provided with a collimating lens system capable of producing a spot no larger than 19mm in a working range of 0 to 150m. The intensity of the beam shall not exceed 5.0mw. The laser tube, electronics, collimating optics, and grade device must be contained inside a waterproof enclosure.

Alignment of the grade instrument shall be controlled by either a sighting scope internal to the laser head and aligned to the beam or a transit placed on line such that line is set into the laser by referencing the transit and laser head to the forward manhole location.

The grade instrument shall initially be adjusted to the proper height to sheet through the pipe by transferring the cut from the engineer's offset hub to a reference point on the grade instrument.

Trench line and grade, also grade for select bedding if required, shall be controlled by observing the laser beam with reference to a mark on a hand shovel, or other suitable target. The laser beam may be placed either at the level of the pipe or above the ground. If placement is at pipe level, each section of pipe shall be placed to line and grade by inserting a pipe target into each pipe and adjusting the pipe until the beam strikes the target at the proper index. If the laser is placed above ground a story pole with an integral plumbing device to intercept the laser beam shall be used.

Line and Grade Control

The engineers will lay out the job by providing an offset hub at each manhole location. The engineers will provide cut sheets for the contractor's use in excavating the work. The cut sheets shall show the amount of cut from hub elevation to pipe invert elevation at each hub. Also, the cut sheets shall show the amount of grade, in per cent, for each section of pipeline.

In executing the work, the contractor shall utilize a laser instrument in conjunction with the engineer's layout work. The grade reading device associated with the laser instrument shall be mounted internally in the system to protect against damage and shall be graduated to read directly to 0.010% grade with interpolation possible to 0.001% grade. The instrument shall be a factory-built, quality-controlled, precision instrument capable of controlling grade to a tolerance of +- 0.01%.

The laser for the grade instrument shall be provided with a collimating lens system capable of producing a spot no larger than 19mm in a working range of 0-150m. The intensity of the beam shall not exceed 5.0 mW.

Alignment of the grade instrument shall be controlled by either a sighting scope internal to the laser head and aligned to the beam or a transit placed on line such that the line is set into the laser by referencing the transit or laser head to the forward manhole location. The grade instrument shall initially be adjusted to the proper height to shoot through the pipe by transferring the cut from the engineer's offset hub to a reference point on the grade instrument.

Trench line and grade, also grade for select bedding if required shall be controlled by observing the laser beam with reference to a mark on a handshovel or other suitable target. The laser beam may be placed either at the level of the pipe or above the ground. If placement is at pipe level, each section of pipe shall be placed to line and grade by inserting a pipe target into each pipe and adjusting the pipe until the beam strikes the target at the proper index. If the laser is placed above ground, a story pole with an integral plumbing device shall be used to intercept the laser beam.

SUMMARY

The use of sewer lasers has become an accepted method for providing line and grade control in sewer construction. Comparing lasers to the older, more traditional methods, there are many benefits.

For the contractor, the benefits of lasers are many:

1. Marked increases in productivity
   - generally in area of 20%-30%, depending on conditions
   - no longer has to set batter boards or string lines
   - each section of pipe goes in faster usually the bedding is correct the first time - only a few seconds are required to check the alignment; traditional methods require several minutes
   - excavated materials can be deposited on both sides of the trench
   - backfilling can be carried out as soon as the pipe is set since there are no batter boards or string lines in the way.

2. Substantial cost savings
   - increased productivity leads to 20%-30% decrease in costs
   - less labour is required - typically one less man per sewer crew
   - save on materials - not as much chance of over-excavating and having to put too much costly fill in place - right amount of bedding can be put in and levelled, the first time.

3. Better utilization of manpower
   - foreman no longer has to help with every section of pipe
   - now he can spend more time on what he should be doing - planning and managing the job.

4. A better, more accurate job
   - use of lasers allows greater accuracy to be attained more easily
   - less costly rework is necessary.

For the inspector, the laser also offers benefits:

1. A better, more accurate job
   - Inspector's job is to ensure just that
   - the finished pipe forms a straight "rifle bore" and a "full moon" can be seen.

2. Less conflict with the contractor to get him to do a good job.

3. Easier to maintain control of job.

SUBJECT:

CONSTRUCTION OF SEWERS WATERMAINS AND ASSOCIATED FACILITIES

TOPIC: 17

EXCAVATION AND BACKFILL

OBJECTIVES:

The trainee will be able to:

1. Recognize various soil conditions and request appropriate modifications in excavation methods as may be necessary for the safe and efficient conduct of the work.

2. Recognize and obtain appropriate compaction densities in backfill materials.

EXCAVATION AND BACKFILL

This section deals with the techniques of excavation of a trench for the purposes of installing a sewer or watermain, as well as the backfill of the same trench after the pipe has been installed.

Pre-Excavation

Before commencing any excavation it is necessary for the inspector to be informed of reasons for the proposed work, and also of its impact on the community.

When working on existing streets it is necessary to be certain that all approvals have been obtained and that all affected parties such as local residents, police, fire and local utilities are informed.

The functional requirements of the project should also be understood, prior to excavation. Some systems are intended to be operational on a block by block basis. Some systems, such as in new subdivisions, will be totally completed prior to operation. Such conditions affect the storing of excavated material, the ordering and placing of construction material should be understood prior to commencing work.

The inspector should insure that the contractor checks for buried utilities by contacting the utility company and requesting a stake out. He should also note overhead wires, trees, etc. as well as those details shown on the plan as it is always possible that something was overlooked at the design stage or was not known. He should establish contact with local residents who may volunteer information regarding old tile drains, abandoned wells, etc.

The inspector should check basement elevations in suspected areas of tight design and look for new structures, poles, and trees which mysteriously appear between original detail survey and final construction.

He should also read the specification pertinent to excavation and backfill thoroughly; and in addition, the soils report. Soils problems are generally predictable and the best time to request an extra pump or extra compaction equipment is before the backhoe digs the first shovelful.

The inspector should make every effort to attend the digging of test holes at the time of tendering to familiarize himself with soil conditions.

Prior to excavation, the most important thing is to check the layout in a general way. the grade sheets should be mathematically correct and the basic reference points verified.

EXCAVATING EQUIPMENT

Selection of appropriate equipment is important. Machines which are too big cause unnecessary damage to trees, poles, etc., and machines which are too small move the job too slowly, which is not only uneconomical but may also cause such problems as settlement due to frozen backfill or side-slope instability.

Bucket widths should also be checked to minimize excavation and maintain side support. Most buckets can be changed very quickly or if necessary an appropriate size can be rented.

Most trench work today is being done by hydraulic backhoes. These provide excellent control; they can be used in most sheathing situations as well as in "V" trench, and come in a variety of sizes.

In very wet tight-sheathed conditions some contractors prefer a clam bucket which is slow but is sometimes a last resort in a deep trench, and may be accepted.

Drag lines generally should be discouraged due to the wide area required for storage of material and considerable resultant damage. Some Contractors prefer drag-lines on wet sandy work but very poor grade control and considerable disturbance of the trench bottom invariably results in a poor job.

Rotary trenchers may be used for shallow, narrow trenches such as powerlines or water services.

Continuous bucket trenchers, capable of depths up to 3m may be used, however delays due to boulders, unseen service pipes and other problems generally cause the contractor to select alternate equipment.

SOILS JUDGEMENT

The inspector should look on every bucket full of dirt as a step into the unknown.

Soil types, water content, slope stability, etc. are constantly changing factors, requiring modifications in method.

Firm clays and till soils below optimum moisture can usually be excavated easily and safely but required careful control during backfill.

Dry sand requires more careful observation during excavation since it can slip or run easily and yet backfilling is comparatively easy.

Dry silt behaves like clay but is not too common and wet silt is all too common and behaves in unpredictable and hazardous ways and often requires special treatment.

Most soils contain portions of sand, silt, and clay and, although it is an oversimplification it is generally true that excessive water and excessive silt are the prime cause of excavation and backfill difficulties.

Danger signals to look for arranged in order of difficultly might be as follows:

Problem	Solution
Free water in trench	Use pumps
Sides tend to slough in	Use box, plus pumps
Bottom tends to move	Use sheathing, pumps, plus special base

STORAGE OF EXCAVATED MATERIAL

It is important to store material so as not to directly damage trees, lawns, etc., but it is also important to judge what might happen during a rain storm or extended exposure to the elements. Many trunk sewers in river valleys are particularly vulnerable if excavated material is stored on the flood plain. A tight operation with minimum exposure is recommended.

Stored material should also be kept back from the wall of the trench to prevent sur-charging, and possible cave-ins, or subsidence cracking. This is a danger signal indicating weak trench bottom and a need for urgent action by the engineer.

EXCAVATION SHAPE

Note the safety requirements as well as the structural requirements pertaining to trench width, since these are sometimes in conflict.

The soil type, water content and stability should be checked regularly and compared with soil classifications under the Occupational Health and Safety Act, which defines the minimum safety requirements for trench side slopes and shoring.

Contractors tend to dig firm soils with trench walls which do not have enough slope and are too narrow and on the other hand, they dig wet silts or free-running sandy soils generally with a tendency to be too wide.

"1.2 metres vertical with one to one slopes" - means just what it says and variations should only be tolerated on the basis of the engineer's direction.

All timber sheathing, metal boxes, air-shore, etc. must be approved by the engineer, but generally all such devices require vertical trench walls. Horizontal bracing will only work at 90° and trying to brace "nearly vertical" walls has caused braces to slip or jacks to jump with fatal consequences.

Generally, all buried conduits are installed in either "embankment" or "trench" conditions. Embankment conditions generally deliver a greater load to the conduit than is the case in trench conditions. This is because the trench sides through friction help in supporting the backfill.

At the point where a trench becomes so wide that the load no longer increases with width the trench is described as having reached the "transition width". This width is measured at the top of the pipe.

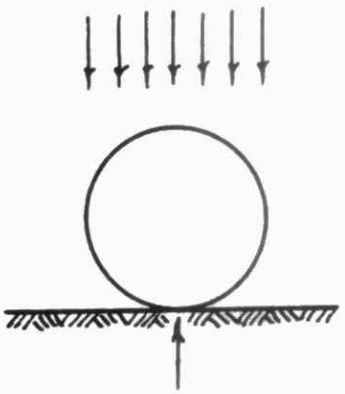
Theoretically a different transition width may be calculated for every pipe diameter, for every soil type and for every height of backfill. The inspector is cautioned against any over-simplification of this concept and it is worth noting that the Concrete Pipe Design Manual lists transition width for all pipes from 150mm diameter to 3600mm diameter requiring 120 charts in the process.

The essential fact to remember regarding trench width is that the load increases with width up to a maximum. If the designer has selected a pipe strength on the basis of an assumed maximum width, it is essential that this width be obtained.

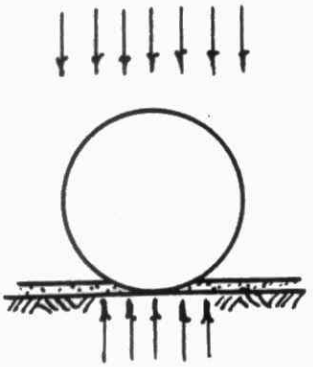
It is also essential that the bedding class be as specified. However, this is more thoroughly covered in the topic "Pipe Installation".

To emphasize these comparisons we refer you to the diagrams identified as Figures 17-1, 17-2 and 17-3.

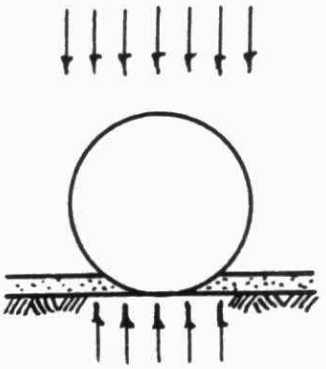
Figure 17-1



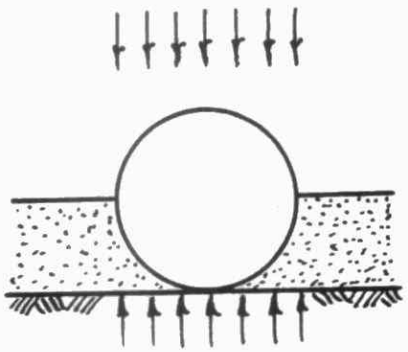
Pipe set on flat foundation without bedding results in high load concentration at the bottom of the pipe.



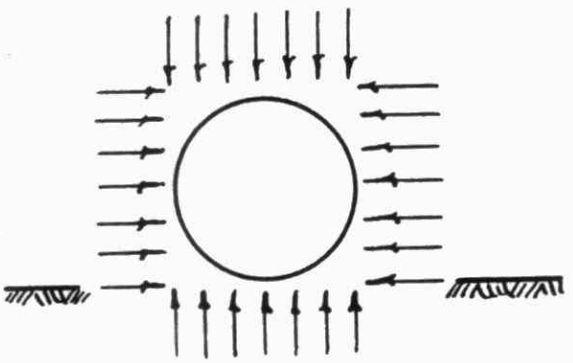
Bedding pipe over 50% of dia. results in 36% increase in supporting strength.



Bedding pipe over 60% of dia. results in 73% increase in supporting strength.

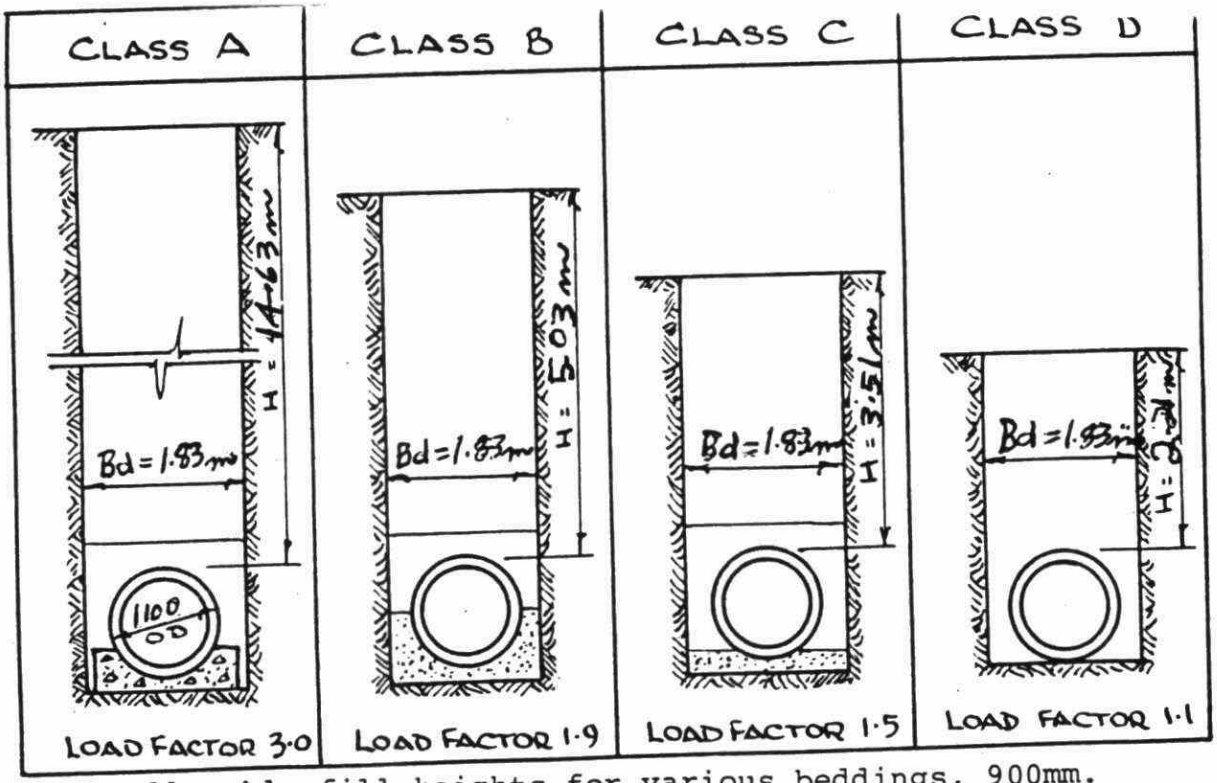


Bedding pipe over 100% of dia. results in 114% increase in supporting strength.



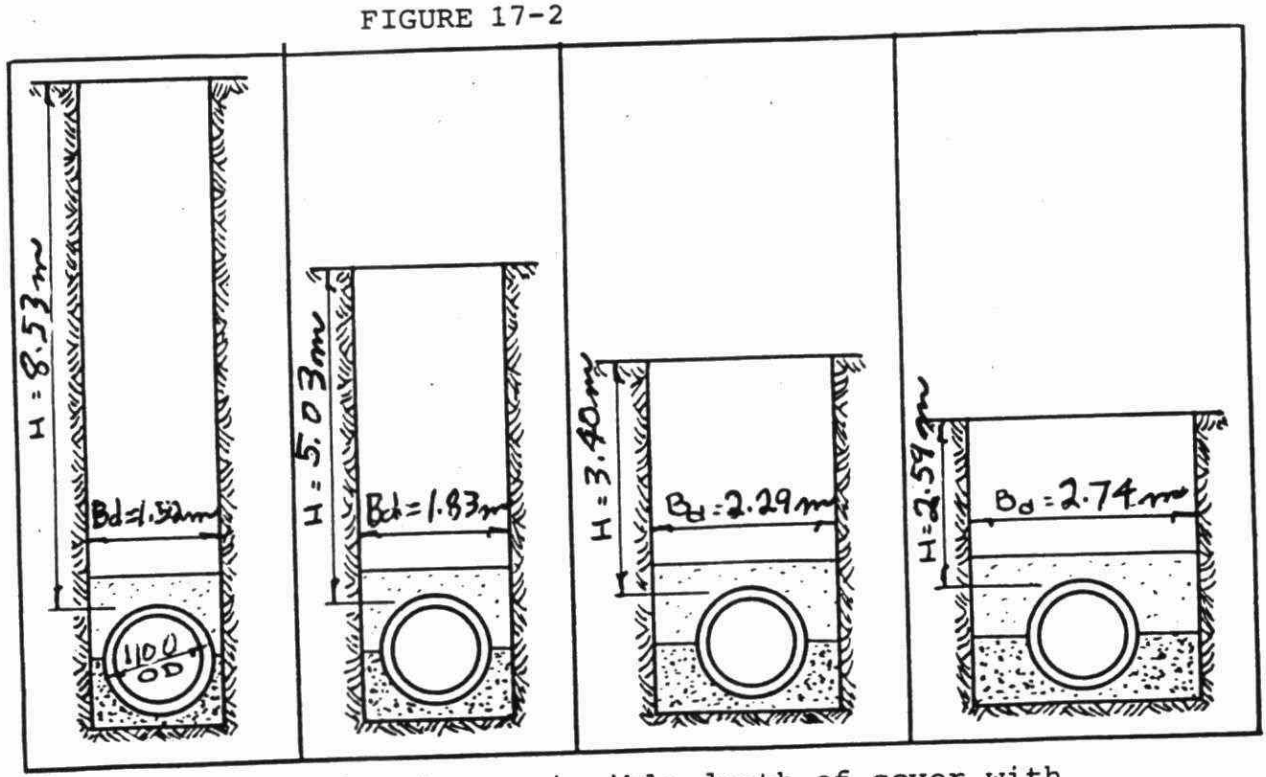
Bedding pipe over 100% of dia. and good sidefill compaction results in 150% increase in supporting strength.

CORRELATION OF BEDDING AND SUPPORTING STRENGTH



Allowable fill heights for various beddings, 900mm. Dia. CSA A257.2 Class III Trench Installation W= 1920 kg/m^3 K= 0.33 μ'= 0.50 (OCPA)

FIGURE 17-2



Variation in permissible depth of cover with increase in width of trench 900mm dia. CSA A257.2 Class III using B bedding W= 1920kg/m^3 K=0.33 μ'=0.50 (OCPA)

FIGURE 17-3

BACKFILL EQUIPMENT

The important consideration in backfilling is to achieve a balanced operation. A small bulldozer or loader feeding a self-propelled vibratory roller is often a good combination. Trying to utilize a machine for backfilling which also doubles as a crane for lowering pipe or for drawing stone or whatever, is a mistake. Backfilling to meet a specified density is the most important part of the work in terms of extra cost to the Contractor if it is not done well.

The inspector should insist on the backfilling being done gently, in layers of a depth within the easy capability of the compactor.

Layers may be sloped, provided the compactor can work on the slope.

It may also be necessary when compacting in sheathed trenches to withdraw the timbers in stages equivalent to the layers of earth being placed. Settlements in such trenches are particularly serious because compaction is difficult and the material tends to bridge to the vertical walls but ultimately a settlement does occur. This usually happens after the Contractor's guarantee has expired, leaving the engineer and his inspector in a vulnerable position.

Types of compacting equipment are many and varied but the following summary of appropriate applications may be useful.

1. Irregular Drum Tampers - clays, tills, silts.
2. Hand Controlled Plate Tampers - sands in shallow lifts.
3. Boom-Mounted Plate Tampers - clays or sands in deep, narrow trenches which preclude the operator's entry into the trench.

Generally speaking all of the above equipment is available in vibratory or static models. A Vibratory type is usually preferable when the moisture content is less than the optimum percentage determined from a "Proctor" test. Static machines usually require more passes, but in over optimum conditions they may be the only practical alternative which will not get stuck.

BACKFILL PROBLEMS

If the Contractor has the best equipment and the best intentions he may still not achieve acceptable results.

Material which is too wet or above optimum moisture presents a much more difficult situation. One approach is to note the "in-situ" density of the wet soil and then try to compact to this density. Another is to insist on a total compactive effort to the extent that no visible voids are present and this may produce tolerable results. However, as soon as moisture content exceeds optimum by more than about 3% it is essential that the inspector have the engineer's approval for whatever concessions may be given. Never act alone.

Frozen ground is also a major problem and if large frozen lumps are included in the backfill it is an absolute certainty that settlement will occur.

Special problem areas also exist near manholes or valve chambers or other obstacles in the ground. Using granular backfill in these areas is one answer but this also requires compaction.

The use of fragmented rock as backfill should be discouraged. However, where no other material is available it is essential to mix sand, gravel, or crushed rock with the rock to fill all voids. It is also essential to create a layer of fine material over the pipe to prevent damage.

Backfilling peat bogs or other very light soils presents a very special conditions. It can be shown that the weight of the pipe is less than the material displaced and, theoretically it should not settle. However, it is necessary to ensure that the weight of the soil being used as backfill does not exceed the weight of the soil removed as settlement may be induced due to compression of the underlying undisturbed peat.

Approaching the Surface

The backfill should not be "topped-off" in the expectation of settlement to make room for specified road base or topsoil thickness.

It is not only neater and cleaner to restore as the job continues, but it also imposes an obligation on the Contractor to obtain good compaction to avoid making up settlements with costly surfacing materials.

Surplus Excavated Material

When surplus excavated material is to be hauled off site, the inspector should check with his supervisor to determine if any monitoring of the disposal of material is required.

SUBJECT:

CONSTRUCTION OF SEWERS WATERMAINS AND ASSOCIATED FACILITIES

TOPIC: 18

PIPE INSTALLATION

OBJECTIVES:

The trainee will be able to:

1. Understand proper methods and procedures for handling, storing, bedding and jointing of pipes.

PIPE INSTALLATION

GENERAL

This topic covers the handling, storage, jointing and bedding of pipe for gravity and pressure pipe lines on municipal-type projects.

HANDLING

The first thing that happens to pipe when it is delivered to a construction site is it is unloaded. However, before the pipe is unloaded it should be checked to see if it is the proper size, class and that it is undamaged (i.e. no chips, cracks, etc.). If the pipe is satisfactory it can then be unloaded. In general it is best to use mechanical means to unload pipe as this protects it from damage. Care should be taken to prevent abuse and damage of the pipe no matter what method of unloading is used since damage unnoticed can result in having to re-lay part of the pipe line due to failure found only after installation (e.g. failure to pass leakage test).

The following are some suggested methods and things to watch out for when unloading pipe on the job.

Concrete Pipe

- "Tailgate" smaller pipe sizes.
- Use a crane on the larger diameters.

Ductile Iron Pipe

Unload using equipment, make sure you don't damage the cement mortar lining (if any) loose and do not drop.

Polyvinyl Chloride and/or Polyethylene Pipe

Avoid severe impact, in particular at temperatures below -10°C.

Avoid undue abrasion, don't slide over truck bed. When using slings they should be rope or webbing not steel or chain.

Vitrified Clay Pipe

Usually shipped in "Cryovac" packed skid loads. Use mechanical means to unload, handle pipe carefully to avoid damage.

Corrugated Steel Pipe

Use mechanical means to unload, and handle pipe carefully to avoid damage.

STORAGE

If pipes are to be distributed along the trench keep the following in mind:

1. Lay or place pipe as near the trench as possible to avoid excess handling later.

2. If trench is open it is advisable to string pipe on the side away from excavated earth wherever possible so that pipe can be moved easily to the edge of trench for lowering into position.

3. If trench is not yet open find out to which side excavated earth will be thrown, then string out on the opposite side (leave room for excavator).

4. Place pipe so as to protect it from traffic and heavy equipment; also, safeguard it from the effects of any blasting that may be done.

5. Care should be taken that pipe do not roll into the trench.

6. Do not place in ditch bottoms.

However, if the pipes are to be stockpiled the following should be remembered.

1. Stockpiles should be built up on a flat base.

2. The bottom layer should be supported uniformly along the barrel of the pipe so that the bells do not touch the ground.

3. When stacking pipe, bells should project over the end of the barrels in alternate layers.

4. Keep various sizes and classes grouped. Put short lengths, fittings, adapters, etc. in separate piles where they will be readily available.

5. When stacking PVC pipe do not stack loose pipe over one metre high and protect from sunlight by covering with a tarpaulin or other opaque material. Be sure to allow for the circulation of air beneath the cover.

6. Pipe should be stacked in a manner so that it will not roll down a grade.

Gasket Storage

Store all rubber gaskets and lubricant at a central point and distribute as needed. Keep them clean, away from oil, grease, excessive head and electric motors which produce ozone. The best way is to leave them in their original cartons and stored in a cool, dry place. Protect from freezing.

JOINTING

General

The main objective when jointing a pipe is to obtain a watertight seal. This is true of both pressure and gravity-flow lines. In this section we will deal mainly with flexible joints for the most commonly used types of pipe.

Concrete Sewer Pipe

Basically, a bell and spigot design with a rubber gasket compressed between the bell and spigot. See Figure 18-1 for various joint types and methods of installation.

Vitrified Clay Sewer Pipe

Flexible joints for vitrified clay pipe shall be as approved by the Ministry of the Environment and as specified by manufacturer.

Joints used for vitrified clay pipe have consisted of the following types:

The "Advanced Design Flex-Lox" joint consists of a smooth resilient polyurethane gasket cast on and firmly affixed to both spigot ends of a plain end vitrified clay pipe and a separate external rigid sleeve made of PVC for joining the pipe together. The preformed resilient spigot components and the internal contour of the PVC rigid sleeve are designed to form a compressed mating surface when the pie is telescopically assembled. See Figure 18-2 for methods of joint assembly.

The "Flex-Lox II" joint consists of a rubber "O-ring" placed in a groove machined in one end of a pipe with a glass fibre reinforced sleeve firmly affixed to the other end of the pipe. The O-ring and the interior of the sleeve form a compressed mating surface when the pipe are telescopically assembled. The method of joint assembly is as follows:

1. Clean the joint surfaces.
2. Install the O-ring in the groove on the spigot.
3. Equalize stretch in the O-ring.
4. Lubricate the outside of the O-ring.
5. Push the spigot end home.

The "Kwik-Seal" joint consists of a separate external coupling in which two rubber gaskets are imbedded. The spigots of plain end pipe are pushed into the coupling forming a watertight seal. The method of joint assembly is as follows:

1. Clean the ends of the pipe and the coupling.
2. Lubricate about 50mm of the end of the pipe to be joined.
3. Place coupling on firm base and press lubricated pipe end into coupling as far as the stop.
4. Lay the first pipe with coupling facing next pipe.
5. Apply coupling on next pipe in the prescribed manner.
6. Lubricate bare end and home into previously laid pipe.

Ductile Iron Pipe

The most common flexible joint is the "Tyton" joint. This consists of a rubber gasket compressed between the bell and the spigot of the pipe. The method of joint assembly is as follows:

1. Clean the gasket groove in the bell.
2. Install the rubber gasket.
3. Lubricate the gasket.
4. Push home the spigot end until the reference mark is reached.
   Mechanical joints are also available.

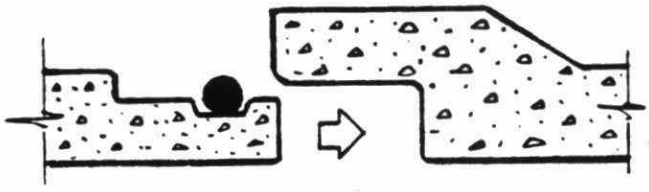
Polyvinyl Chloride Sewer and Pressure Pipe

The joint used on PVC pipe is usually similar to the "Tyton" joint used o n ductile iron pipe. The only installation difference is that rather than lubricating the rubber gasket the spigot of the pipe is lubricated before homing the pipe to the reference mark. Mechanical joints such as uni-flange are also available.

Polyethylene Pipe

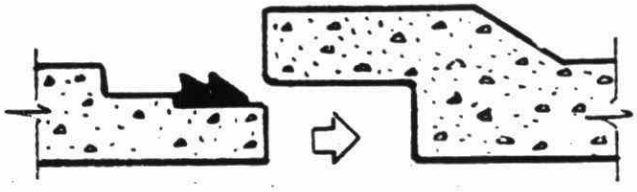
Butt fuse according to manufacturers specifications and using approved equipment and tools.

Figure 18-1 Typical Joint Designs



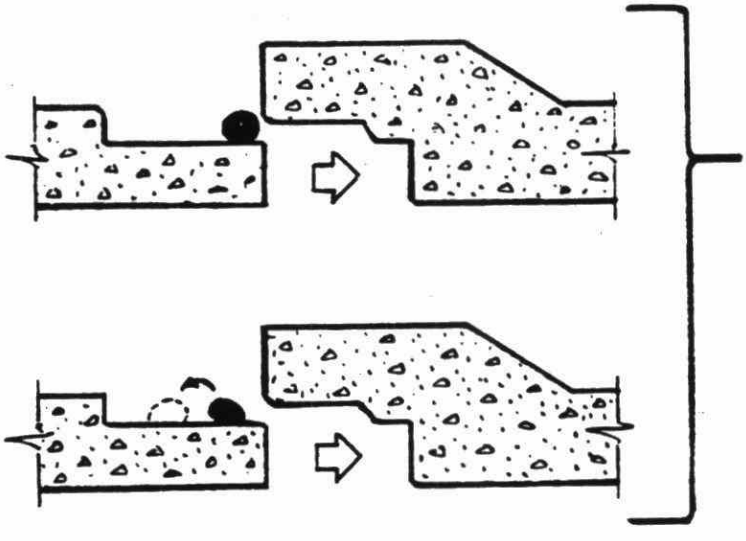
RECESSED

CLEAN JOINT SURFACES, LUBRICATE INSIDE OF BELL, LUBRICATE "O" RING, EQUALIZE TENSION.



SINGLE OFFSET

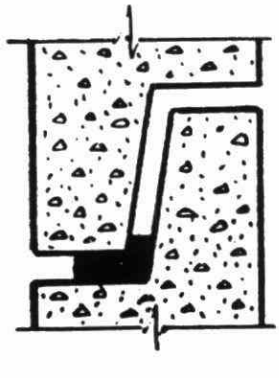
CLEAN JOINT SURFACES. POSITION GASKET ON SPIGOT & EQUALIZE TENSION. APPLY LUBRICANT ON TOP OF GASKET & INSIDE OF BELL.



ROLL ON

CLEAN JOINT SURFACES. POSITION GASKET AT END OF SPIGOT.
DO NOT LUBRICATE

JOINT OPENING — FOR PIPE UP TO & INC. 1350 mm — 13 mm MAX.
FOR PIPE 1500 mm & LARGER — 19 mm MAX.



MANHOLE

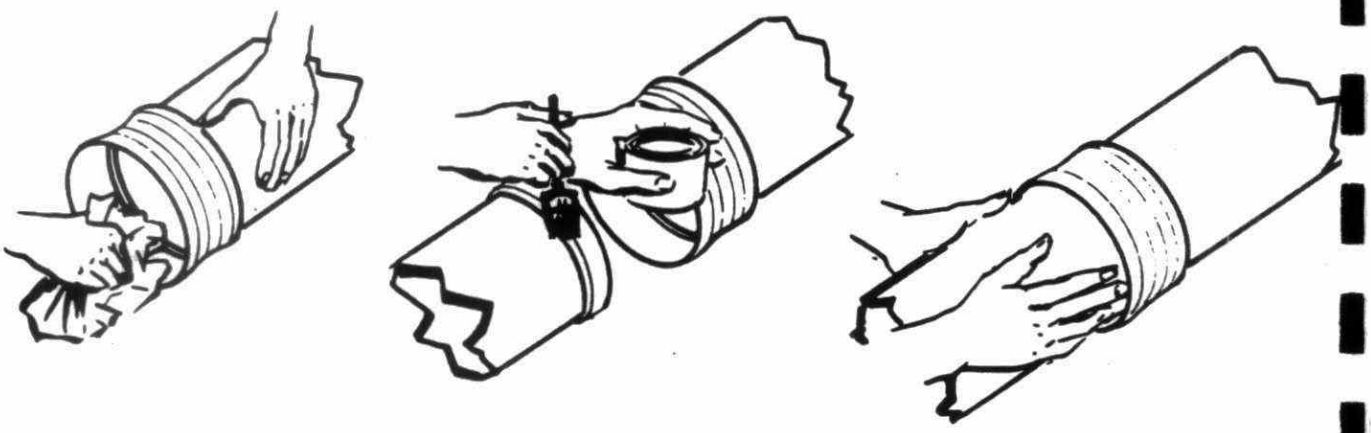
CLEAN JOINT SURFACES
NO LUBRICANT REQUIRED

Figure 18-2 Installation Instructions

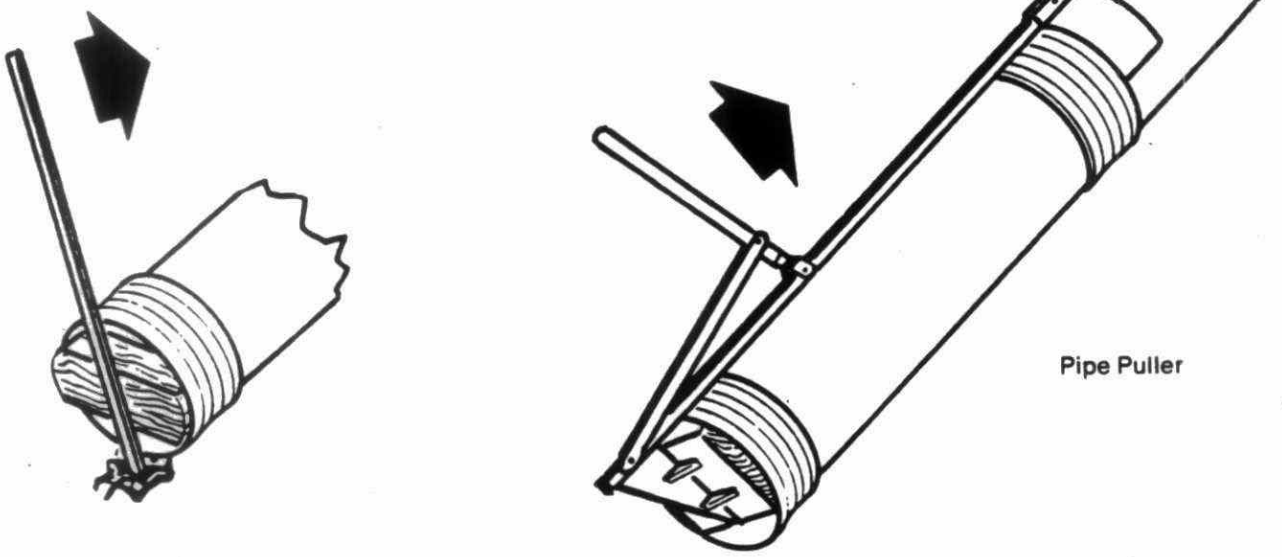
① Wipe the PVC collar and spigot clean

② Lubricate spigot

③ Insert spigot firmly into PVC collar



④ Complete the assembly with the use of a pry bar or, where sewer is laid in solid rock, the pipe puller (illustrated) available from the manufacturer.



Pipe Puller

When using a pry bar it is necessary to use the Flex-Blok or similar timber fulcrum.

NEW "FLEX LOX" VITRIFIED CLAY SEWER PIPE

National Sewer Pipe Ltd.

Principal Considerations for Jointing

1. Always clean the joints and gaskets prior to jointing.

2. Keep the pipe horizontal when jointing (use slings or cranes).

3. Use a bar or come-along when homing pipe. Never use the bucket of a backhoe as it may break the pipe.

4. Apply lubricant evenly to gaskets and pipe (where applicable).

5. When using a trench box anchor pipe so they will not move when the box is pulled ahead.

6. Obtain manufacturer's jointing instructions and follow them.

BEDDING

General

The purpose of bedding a pipe is two-fold; 1) to provide firm continuous support for the pipe to maintain the grade set for the pipe line; and 2) to provide the soil side support for the pipe to enable the pipe and the soil to work together to meet the design load requirements.

If a pipe is set on a flat foundation without any bedding there is a high load concentration at the bottom of the pipe. If the pipe is bedded over 50% of the diameter of the pipe, the pipe can support 36% more weight than it could without any bedding. Increase the bedding to 60% of the diameter the pipe can now support 73% more weight than with no bedding. Extend the bedding to 100% of the pipe diameter and the pipe will now support 114% more weight than the original unbedded pipe. Add good side fill compaction to bedding under 100% of the pipe diameter and you now have 150% of the load-carrying capacity that you had when you started with an unsupported pipe.

Classes of Bedding

There are four classes of bedding commonly used - classes A, B, C and D. The best class of bedding (Class A) enables a given pipe to carry a load three times as much as that load the pipe could carry in a three-edge bearing test. The worst class of bedding (Class D) enables the pipe to carry little more than it would in the three-edge bearing test. This ability of a bedding to increase the load-carrying capacity of a pipe over its load-bearing capacity in the three-edge bearing test is measured by a term called the load factor. If a certain bedding allows a given pipe to carry twice as much load as it can carry in the three-edge bearing test, the load factor is 2.0. Similarly, if a bedding allows a pipe to carry 3.0, 1.5 or 1.1 times as much load as that pipe in the three-edge bearing test then that bedding would have a load factor of 3.0, 1.5, or 1.1 respectively.

Class 'A' Bedding

Generally specified as a concrete cradle under the pipe extending approximately half way to spring line. Its load factor is 3.0.

Class 'B' Bedding

Generally specified as a well compacted granular cradle extending out to the walls of the trench and up to the spring line of the pipe. Its load factor is 1.9.

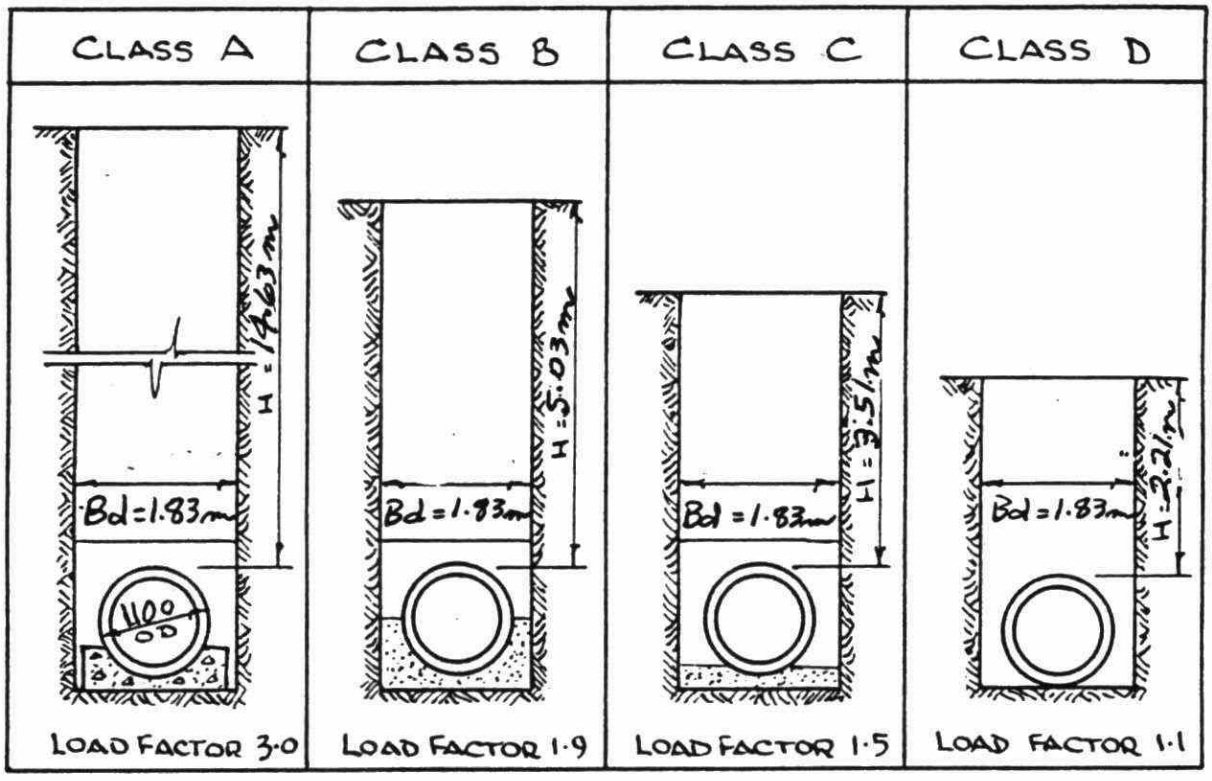
Class 'C' Bedding

Generally specified as shaping the bottom of the trench to receive the pipe. Its load factor is 1.5.

Class 'D' Bedding

Basically burying the pipe with no preparation of the trench bottom. Its load factor is 1.1.

The following figure illustrates the effect of the various bedding types upon the depth of cover over a pipe with the only variable the bedding type.



OCPA

Figure 18-3

Allowable fill heights for various beddings, 900mm. Dia. CSA A257.2 Class III Trench Installation W=1920kg/m^3 K=0.33 ù=0.50

Bedding Procedure

The primary requirement when bedding a pipe is to have a stable and uniform foundation. It should be smooth and free from large stones, rock, boulders, construction material debris, large clods of earth and frozen material.

A 75 to 150mm layer of bedding material is usually spread on the original trench bottom in order to bring the bedding up to grade for laying the pipe. The pipe barrel must be given even continuous bearing for its full length. There should be no high or low spots on the bottom of the trench nor should couplings or bells rest on the ground. The above cause point loads which can result in breaks in the pipe. The high and low points can be avoided by levelling the trench bottom and digging holes under the bells or couplings if necessary.

Once the trench bottom has been shaped properly and the bell holes dug it is necessary to compact the material under the haunches of the pipe. This is a critical part of the operation. Usually this is done with hand tampers. In large diameter pipe it is usually best to shape the trench bottom to fit the O.D. of the pipe. After the material under the haunches of the pipe has been satisfactorily, bring the rest of the bedding up to the required height in shallow lifts that are properly mechanically compacted.

Bedding Material

The best bedding material is a well graded granular material up to 19mm in size free from lumps and frozen material. In rock trenches it is best to use a 19mm clear stone so that the bedding material cannot wash away down fissures in the rock which could happen with a bedding material containing fines.

Principal Considerations for Bedding

1. Use 19mm maximum size granular material.

2. When bedding plastic pipe, it is paramount that bedding material is properly compacted out to the trench walls since a flexible pipe uses the strength of surrounding earth to help carry the load.

3. When installing concrete cradle (Class 'A' bedding) in a rock cut make sure there is a space between the walls of the trench and the concrete for an "insulating" material.

4. Don't use bedding material with "fines" in a trench cut through rock.

SUBJECT:

CONSTRUCTION OF SEWERS WATERMAINS AND ASSOCIATED FACILITIES

TOPIC: 19

DISINFECTION OF WATERMAINS

OBJECTIVES:

The trainee will be able to:

1. Recall the operations involved in the disinfection procedure of newly constructed watermains.

2. Identify the methods of watermain chlorination.

DISINFECTION OF WATERMAINS

GENERAL

In a public water-distribution system, all newly laid watermains, or existing watermains which have been repaired, should be disinfected before being placed into service. It is necessary to follow this practice in order to protect the consumers against the possibility of infection which could result from ingestion of water contaminated by disease producing organisms.

In all likelihood, the newly constructed system will have sustained contamination during transit, storage on the site of the components and laying of the piping. Often the pipe must be laid in soggy trenches and possibly on occasions be in contact with wastewater or even sewage admitted into the trench through service cuts.

In general, the disinfection procedure consists of four operations.

1. Cleaning
2. Chlorination
3. Final Flushing
4. Bacteriological Testing

CLEANING

To ensure successful disinfection, the pipe-line system must be mechanically cleaned. Lumps of mud, sand, dirty water or a variety of debris left in the piping during construction, if allowed to remain in the system, will shield bacteria from contact with the chlorine solution resulting in incomplete disinfection and possibly delivery of contaminated water to the consumers. Improper cleaning of new watermains may also cause problems, such as:

1. Growth of nuisance organisms in the distribution system creating bad taste, odours or discoloured water.

2. A dirty new watermain may contaminate the existing water distribution system.

Watermains may be successfully cleaned by a method commonly referred to as "foam swabbing". Flushing or cleaning of mains by any method should be done only with potable water. This condition also applies to pressure testing of the system.

DISINFECTION

At the present time chlorine is the only disinfecting agent in use for disinfection of watermains. Although there are other bactericidal agents which would be satisfactory for disinfection purposes, to date their use has not been generally accepted. Chlorine may be introduced into watermains by the use of the following methods:

1. Gas Chlorination
2. Hypochlorination

Gas Chlorination

A gas-water mixture is fed into the line with a solution feed chlorinator. Dry gas may be fed directly into the main using proper equipment to regulate the rate of gas flow and provide effective diffusion of gas into the water which is simultaneously entering the pipe. It is very important that, in both cases, the feeding arrangement must provide for preventing backflow of water into the chlorine cylinder.

Hypochlorination

A solution of water and chlorine bearing compounds is most commonly used for disinfection of watermains, because of the relative ease and safety with which these solutions could be handled and applied. Three of the most common chlorine bearing compounds which are readily available as commercial products and having a known chlorine content are:

1. Calcium Hypochlorite, Trade names - "HTH", "Perchloron" and "Pittchlor" contains up to 70% available chlorine.

2. Chlorinated Lime or "bleaching powder" contains approximately 33% available chlorine.

3. Sodium Hypochlorite a variety of liquid solutions containing between 5% and 14% available chlorine.

These compounds are normally fed as a water mixture of one per cent solution.

APPLICATION PROCEDURES

Continuous Feed

The chlorine, fed either by a gas feeder or a hypo feeder, is introduced into the watermain through a corporation stop at the top of the pipe at the start-point of the new line. Simultaneously, water from the existing distribution system or other suitable (approved) source is introduced into the pipeline undergoing disinfection. The flow of water and chlorine mixture should be fairly accurately proportioned so the chlorine dosage will be no less than 50 mg/l. subsequent to filling the pipeline, the chlorinated water should remain in it for at least 24 hours. During this time all new valves and other appurtenances within the system should be operated to expose all components to disinfection. A chlorine residual test should be done at a number of selected locations to ensure that the strength of residual chlorine is 25 mg/l in the system at the end of the 24 hour period. If the residual chlorine is below 25 mg/l at the end of the contact period of 24 hours, the contact period must be repeated with a new solution of 50 mg/l starting strength.

Under certain conditions it may not be feasible to retain the chlorinated water in the pipeline for 24 hours. In such cases, by increasing the chlorine concentration, the retention time may be decreased. This practice, however, should be carried out with consultation and agreement of the local health authority or other agency responsible for proper disinfection of the system. High concentrations of chlorine, 100 mg/l or higher, should be used only in emergencies because of the possibility of damage by highly corrosive chlorine to the pipe, valves, hydrants and other appurtenances.

Slug Method

This method is suitable for use with mains of large diameter for which, because of the volumes of water involved, the continuous feed method is not practical.

Water from the existing distribution system or other approved source of supply shall be made to flow at a constant, measured rate into the newly laid pipeline. The water shall receive a dose of chlorine, also fed at a constant, measured rate. The two rates shall be proportioned so that the concentration in the water entering the pipeline is maintained at no less than 300 mg/l. The chlorine shall be applied continuously and for a sufficient period to develop a solid column or "slug" of chlorinated water that will, as it passes along the line, expose all interior surfaces to a strength of the chlorine solution should be checked at outlet valves near the upstream ends of the line by chlorine residual measurements.

As the chlorinated water flows past tees and crosses, related valves and hydrants should be operated to disinfect these appurtenances.

Flushing

Following the prescribed retention period, the chlorinated water should be removed from the mains by proper flushing and the piping filled with clean potable water of proven quality. This procedure should be carried out with the assistance of the local P.U.C. and the Health Authority.

BACTERIOLOGICAL TESTING

After the new works have been adequately chlorinated and filled with clean fresh water, a prescribed number of water samples should be collected at predetermined locations and submitted to a laboratory for bacteriological examination. When the laboratory report has confirmed the water in the new system to be bacteriologically satisfactory, it may only then be placed into service to the consumers, but never before. Should the laboratory results prove the water to be bacteriologically unsatisfactory, the disinfection of the system must be repeated.

When collecting samples, care should be exercised to ensure the sample is not contaminated because of unsanitary sampling techniques. Both the chlorination of the new works and the collection and submission of samples should be carried out by competent and reliable individuals; for example by personnel from the local Health Authority. If these procedures are carried out in a careless manner, it may be necessary to repeat the chlorination process resulting in increased cost and loss of time.

SUBJECT:

CONSTRUCTION OF SEWERS WATERMAINS AND ASSOCIATED FACILITIES

TOPIC: 20

LEAKAGE & PRESSURE TESTING OF GRAVITY SEWERS, SEWAGE FORCEMAINS, AND WATERMAINS

OBJECTIVES:

The trainee will be able to:

1. Perform infiltration and exfiltration tests on sewers.
2. Perform pressure and leakage tests on watermains and sewage forcemains.
3. Recall the procedure and the safety requirements for the low-pressure air test.
4. Recall some methods of leak detection.

LEAKAGE AND PRESSURE TESTING

LEAKAGE TESTING OF SEWERS

Sewers are tested for leakage both in and out of the pipeline. Leakage into a system is known as infiltration and leakage out of a system is known as exfiltration.

Infiltration of ground water into a sewage system is undesirable because:

1. It reduces the hydraulic capacity of the pipe to carry wastewater.

2. It overloads the treatment facility resulting in either insufficient treatment or requiring capital replacement.

3. It allows roots to enter the system which cause blockages.

4. It washes fine material from the bedding into the pipe resulting either in structural failure of the pipe or collapse of the ground above due to settlement.

Infiltration results in increased cost because it is found in the fact that it is often necessary to expand either the collection or the treatment system much earlier than was planned.

Exfiltration from a sanitary sewage system is undesirable for two reasons. Sewage flows must maintain a minimum velocity of 0.6 m/sec in order to prevent the solids from settling out and eventually blocking the sewer. It is therefore undesirable to lose the transporting medium (i.e. water by leakage). The other factor is of course that the leakage may eventually pollute the ground water table in that area.

Inspectors cannot watch construction crews every minute to ensure good work. Even if they could, damage may occur during the backfill and compaction of the pipeline. Since every pipeline, no matter how well laid, leaks (due to joints, pipe wall porosity, etc.), standards of acceptable infiltration and exfiltration have been set down by the Ministry of the Environment. The standard for leakage is that the leakage shall not exceed 0.075 L per mm internal diameter of pipe per 100 mm of pipe per hour.

Example: The distance from one manhole to the next manhole is 100 m, the internal diameter of the sewer is 250 mm. The allowable infiltration or exfiltration is therefore:

$$0.075 \times 250 \times 1 = 18.75 \text{ L/hr. or } 18.75 \times 24 = 450 \text{ L/day}$$

Infiltration Test (Figure 20-1)

Every section of sewer where the ground water table is 0.6 m or more above the crown of the highest pipe in the section is to be tested for infiltration. The testing procedure is as follows:

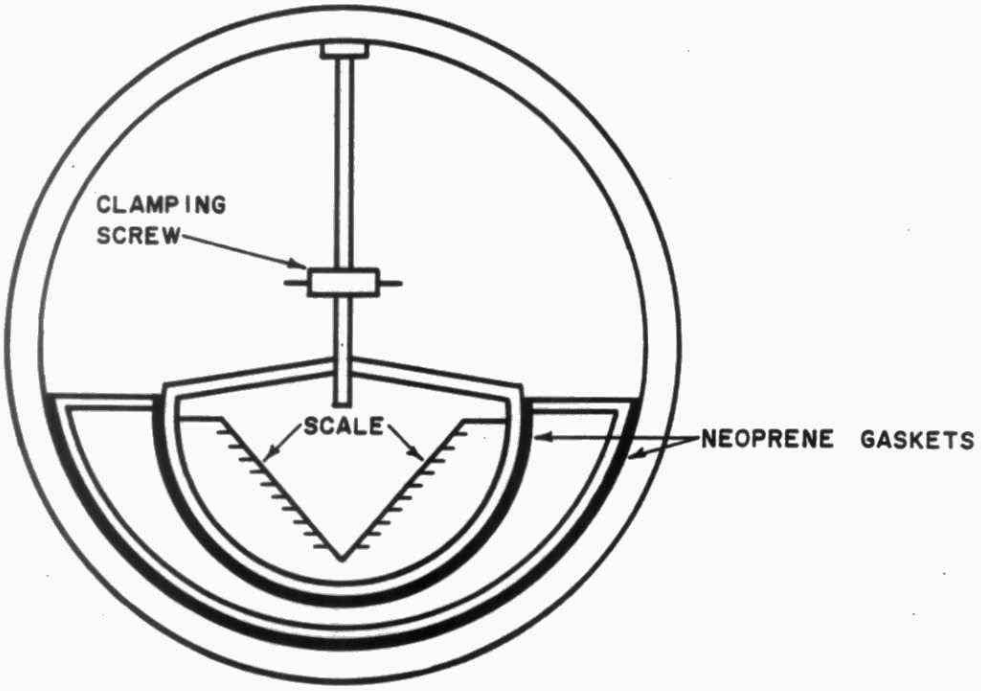
1. Block off the upstream manhole with a pipe plug.

2. Using a V-notch weir; install weir in lower end of section to be tested and allow water to flow over the weir until the flow is steady, without fluctuations. Read the flow from the weir (usually weirs are graduated in litres per day). Record the flow reading, as well as the date, sewer diameter, distance from manhole to manhole, manhole numbers and the approximate height of ground water above the crown of the highest pipe.

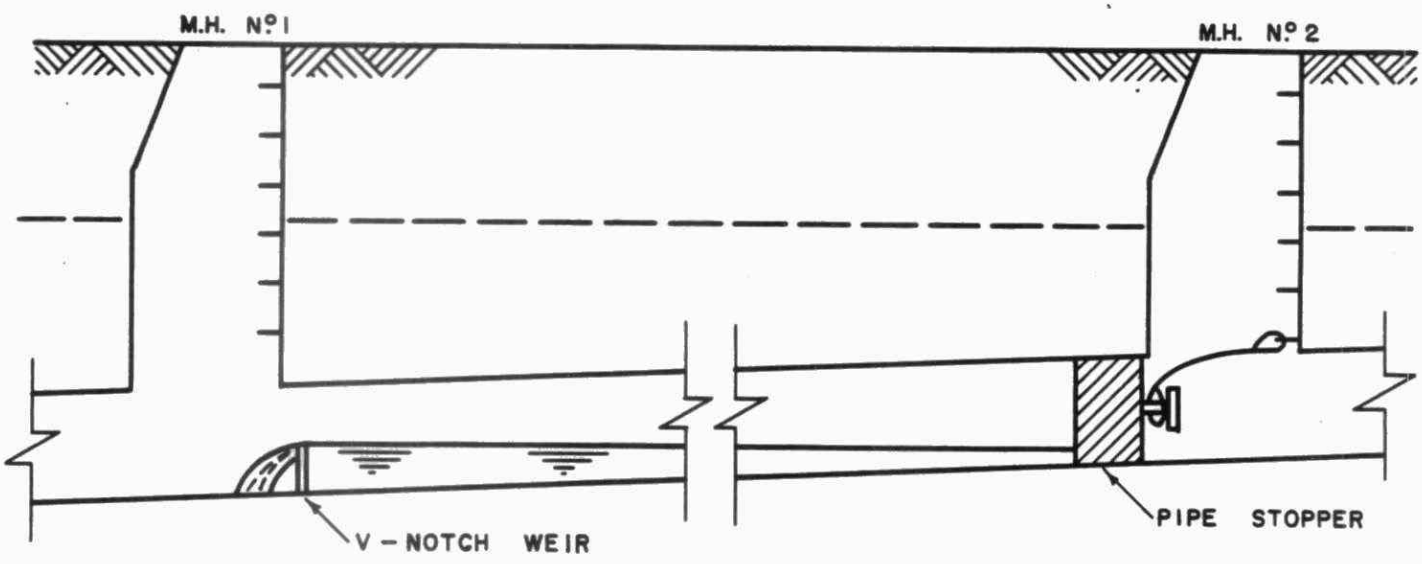
3. An alternate to the use of the weir is to block the exist from the lower manhole and allow the water to build up in that manhole. Pump the water periodically into containers of known volume. This method should be done over a much longer period of time since it is somewhat more inaccurate.

4. Calculate leakage and compare to allowable leakage based on 0.075 L per mm of internal diameter of pipe per hundred metres of pipe per hour. Service connections from the mainline sewer to the property line must be included in the calculation of the allowable leakage.

Figure 20-1



V - NOTCH WEIR



INFILTRATION TEST

Exfiltration Test (Figure 20-2)

In testing for exfiltration, the minimum net internal head on the section of sewer being tested shall be 0.6 m measured from the highest point of the highest service connection in the said section, provided that the maximum net internal head on the line is not greater than 8 m. In calculating the net internal head, allowance should be made for the applicable ground water head, if any. The method of testing is as follows:

1. Install a plug at the lower end of the test section.

2. Plug the incoming pipe(s) at the upper manhole.

3. Fill the section with water at the upper manhole to a level 0.6 m above the highest pipe in the section being tested. Do not exceed a net internal head of 8 m. Do the filling slowly so that air can escape.

4. Allow the pipe and manhole walls to saturate with water for several hours, then fill to the level required in Step 3. Measure the distance from the water level to the top of the manhole frame.

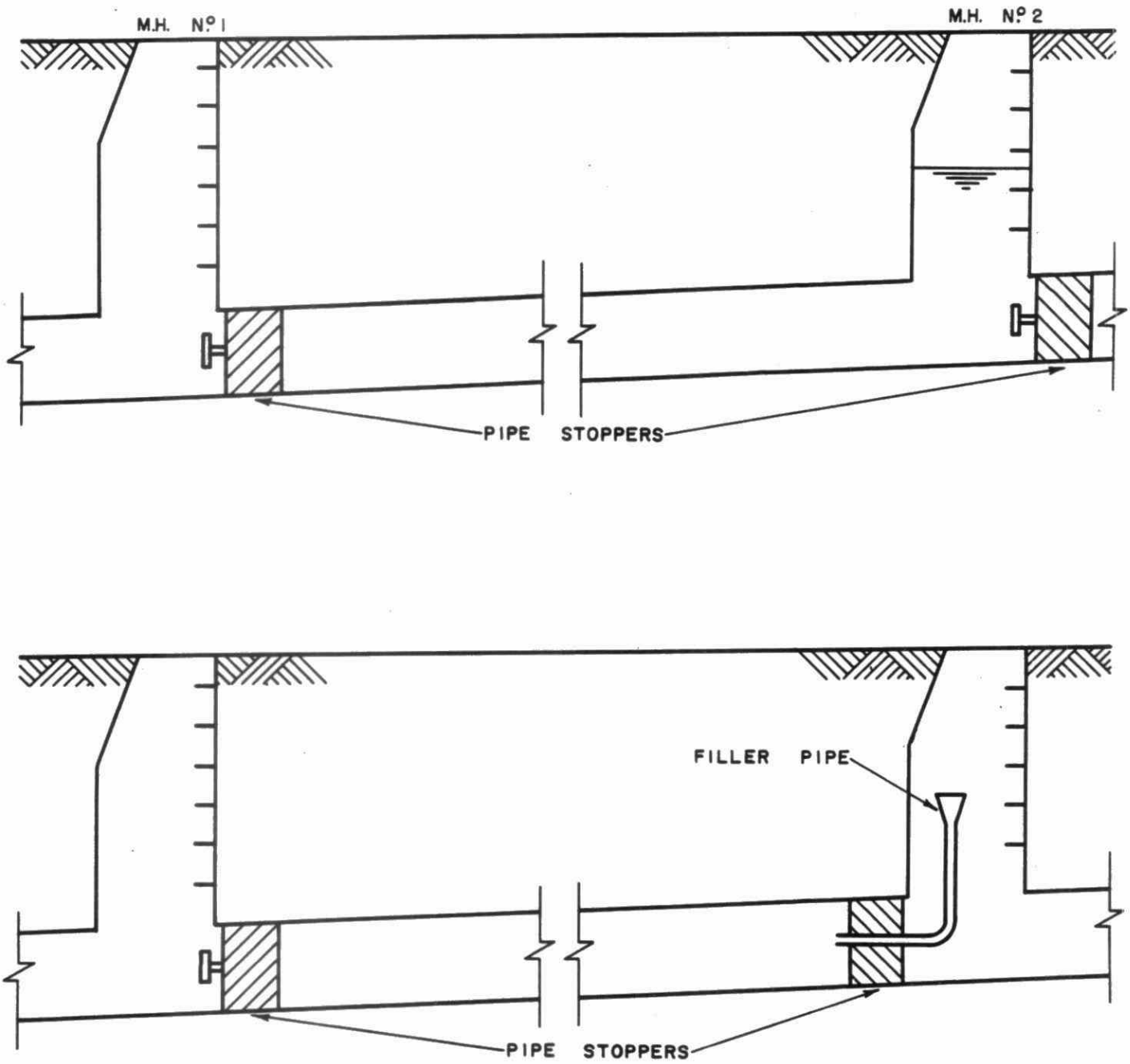
5. Let stand for one hour and then re-measure the depth of water from the top of the manhole frame to the surface of the water. Therefore, this is the exfiltration in the section including the upper manhole.

6. Calculate the amount of exfiltration by means of volumes and compare it to the allowable exfiltration.

7. Record the amount of exfiltration and other pertinent data as in Step 2 for Infiltration.

8. The allowable exfiltration is 25% greater than for infiltration. In calculating the allowable exfiltration, include all service connections and an allowance of 3.0 litres per hour per metre of head above the invert of the manhole for each manhole included in a test section.

Figure 20-2



EXFILTRATION TEST

General Notes for Testing

1. Sewers should be tested immediately after completion.

2. Testing must be done before the private portions of the service connections to the houses are installed.

3. Ensure that installed risers are properly plugged so that no leakage can occur there.

4. When testing for exfiltration ensure that the total head at the lower manhole does not exceed 8 m.

5. Exfiltration testing is used whenever there is less than 0.6 m of external head on the highest pipe in the section to be tested. In all other cases, an infiltration test is used.

Low Pressure Air Testing

An alternative to the above two methods of test is low pressure air testing. Low pressure air testing is best used where testing by water is difficult due to water not being readily available or because of freezing problems.

It can be also used in lieu of exfiltration testing when the net external head on a system exceeds 8 m. However, there is no accepted data relating actual leakage allowances to air pressure testing.

Test Procedure (Figure 20-3)

The low pressure air test is a test which determines the rate at which air under pressure leaves an isolated section of a pipeline. This rate indicates the presence or absence of leaks. The test procedure follows:

1. Isolate Pipe to be Tested

   The section of pipe between two manholes to be tested is plugged at each end. Plug the ends of all branches, laterals and wyes which are included in the test section. All plugs should be carefully braced to prevent slippage and blow-out due to internal pressure. One of the plugs provided must have an inlet tap or other provision for connecting an air hose.

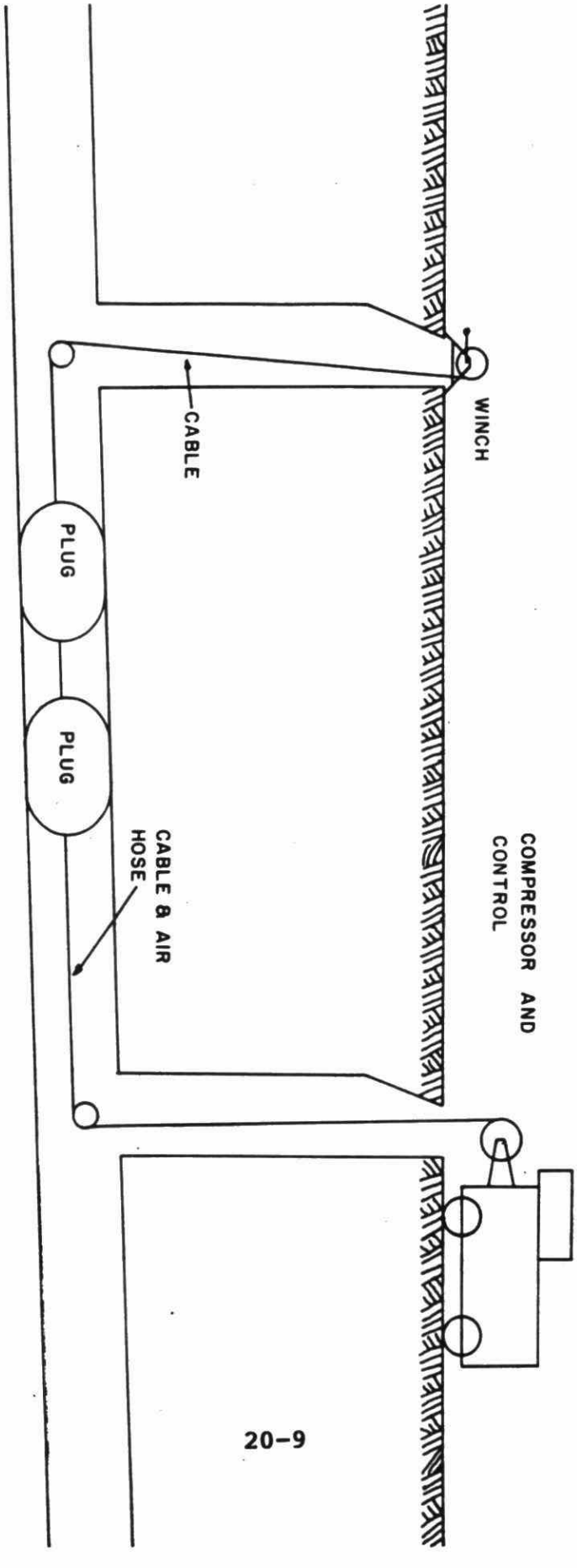
2. Connect Equipment

   Connect one end of an air hose to the plug used for the air inlet. Connect the other end of the hose to the portable air-control equipment. This equipment consists of valves and pressure gauges needed to control the rate at which the air flows into the test section as well as providing a means of monitoring the air pressure inside the pipe. Connect an air hose to a source of compressed air and the control equipment.

3. Add Air

   Supply air to the test pipe section. Monitor the air pressure so that the pressure inside the pipe does not exceed 34.5 kPa.

Figure 20-3


WINCH
CABLE
PLUG
PLUG
CABLE & AIR HOSE
COMPRESSOR AND CONTROL

4. Stabilize

When the pressure reaches 24.1 kPa, throttle the air supply so that the internal pressure is maintained between 20.7 and 24.1 kPa for at least 5 minutes. This 5 minute period allows time for the temperature of the air to come to equilibrium with the temperature of the pipe walls. During this time, check all of the accessible plugs with soap solution to detect any possible leakages. If any plugs are found to leak, bled off the air, tighten the plugs, and begin again.

5. Determine Rate of Air Loss

This step can be performed in two ways.

a) Flowmeter Method - this procedure is used when the rate of air loss is high (greater than 4.7 L/sec.). The control equipment includes rotameters or other air flow measuring devices. Air is supplied to the pipe section at such a rate that the internal pressure is maintained at 20.7 kPa. The flow rate is read in L/sec. The pressure of the metered air is read. The rate of air flow is corrected for pressure and temperature and is recorded in L/sec. of air under standard conditions of pressure and temperature. (Standard pressure is 101.3 kPa, standard temperature is 20°C.

b) Stop-Watch Method - this procedure is used for the testing of newly constructed pipe lines where air losses are usually small. the control equipment consists of pressure gauges, valves and a pocket stop watch. After the temperature has been allowed to stabilize for the five-minute period, the air supply is disconnected, and the pressure is allowed to decrease to 20.7 kPa. At 20.7 kPa, the stop-watch is started to determine the time required for the pressure to drop to 17.2 kPa. The time required for a loss of 2.5 kPa, at an average pressure of 20.7 kPa can be used to compute the rate of air loss. For a precise calculation of the rate of air loss, the temperature of the air inside the pipe section and the barometric pressure should be determined and recorded. However, this refinement has been found unnecessary for the usual acceptance test.

Safety Requirements

The use of air under pressure for the testing of sewers has added two new hazards which are of concern to the users of this method.

It is very important that the plugs be properly secured and that care be exercised in their removal. The total force on a 325 mm plug at 0.31 kPa is over 3500 N. If one considers that a plug weighing 4.5 kg, acted upon by this force over a distance of 0.3 m, attains a velocity of nearly 22m/sec or almost 80 km/hr., it is obvious that the pressure in the pipe must be completely relieved before the plug is loosened. This fact has usually precluded the use of compressed air for the testing of larger pipes and pipelines of great length. The second hazard is the possibility of loading the sewer line with the full pressure of the compressor. It is interesting to speculate as to what would yield first if 1000 to 1400 kPa pressure were imposed on a sewer line.

> *Under no circumstances should a person be allowed to be inside a manhole while the air pressure is applied to the pipeline.*

In some sets of equipment the piping is arranged so that the pressure reducer and flowmeters can be by-passed by an appropriate manipulation of the valves. If for any reason the valve is not closed at the proper time, the sewer line could be exposed to very high pressure. It is therefore advisable that all air testing apparatus be equipped with a pressure relief device designed to blow out at 40-55 kPa.

General Comments

Testing sewers by using air has been opposed in the past partly for the lack of reliable formulas, experience and partly for lack of less porous pipe materials. Since then sewer materials and construction practices have improved to the point that air testing is now practical. However, it has to be emphasized that the Contractor's representative on the site as well as the Inspector have to be thoroughly familiar with this testing method and its hazards. Air testing is not recommended for use on concrete sewer pipe.

Ground Water Adjustment

An air pressure adjustment is required when the prevailing ground water is above the sewer line being tested. Under this condition, the air test pressure must be increased 9.8 kPa for each metre of water above the invert of the pipe.

LEAK LOCATION IN NEW SEWERS

An infiltration, exfiltration or low pressure air test only indicates whether a section of sewer is satisfactory of unsatisfactory. If a section of a sewer has failed the above tests, it now becomes a question of exactly locating the faults so they can be repaired to give a watertight system. There are two common methods used to locate leaks in a system, they are air-testing equipment and camera inspection.

Air testing equipment can be very effectively used for the location of leaks in a section of sewer that has failed to pass a leakage test. This is done by connecting two inflatable test plugs with a short length of cable and air hose and testing the line in short sections by pulling the connected plugs through the sewer. The plugs are inflated, pressure built up, and the test made. The plugs are then deflated and moved to the next position and the test is repeated until the leak is located. By using this method, it is possible to cover about 60 m of sewer per hour and locate leaks fairly accurately. The test sections should overlap each other to prevent covering a leak with a test plug.

One drawback to the above method is that it does not identify what the problem is - a broken or crushed pipe, loose joint, poor service connection or displaced gasket. Camera inspection, using either a still or TV camera, solves this problem. In this method, a camera is pulled through the pipe from manhole to manhole by means of a previously strung through cable.

The still camera method uses an automatic 35 mm camera and electronic flash contained in a watertight casing. A power cable leads from the camera to control equipment located on the ground surface which is used to remotely control the picture taking. At the opposite end to the power cable is a normal wire cable used to pull the camera through the pipeline. Normally, a camera is winched forward into the pipe a desired distance (usually about 1.5 m and then stopped). The operator then trips the camera - the shutter and flash operating together by remote control. Chainages into the pipe are determined from graduations on the power cable which is trailed behind. It is possible to locate faults within 30 mm by careful interpretation of the resulting photographs.

Camera inspection is also performed using a TV camera encased in a watertight casing. This method is more versatile since it is possible to see the complete sewer without missing any parts. A continuous picture of the sewer is projected onto a TV monitor, usually located inside of a specially equipped van. If an interesting feature appears, the camera can either be backed up or pulled ahead to get a close-up view. If desired, still photographs can be taken off the monitor screen for a permanent record. However, some systems employ an automatic still camera located in the same casing as the TV camera. This latter method gives better photographs for interpretation. It is also possible to record the results of the TV inspection on video tape.

Photo inspection of a system should not be substituted for some sort of leakage testing. It is not possible for TV or still inspection cameras to look up service connections and exfiltration leaks located there are missed. Nor is it practical to put a camera through a sewer service connection pipe.

Inspection cameras can also be used to evaluate the condition of sewers that are in service. From the results of the inspection it can be determined whether to re-build or rehabilitate the sewer.

ALIGNMENT CHECK OF SEWERS

Once a sewer has been built it should be checked to see whether it has been laid true to line and grade. Basically this means looking through the sewer. The downstream end of the sewer should be "backlighted" with a powerful light and one should look downstream, either directly or through the reflection in a mirror to see if the pipe is off line or grade. The normal use of inspection cameras does not tell one the elevation of the sewer between manholes. At the best, a rough indication of dips and misalignment may be obtained from depth of ponded or back-up water.

TESTING OF WATERMAINS

Once a watermain has been thoroughly cleaned it should be tested for water tightness. This will give an indication of the quality of workmanship and materials used in the watermain and its fittings. Normally, the main is tested for both pressure and leakage.

The normal test pressure is 1 MPa, (1000 kPa) which is well above the maximum working pressure of a water system.

However, care must be taken, when testing plastic pipe, that the pipe is not over stressed. Polyvinyl chloride (PVC) pipe must be tested to a pressure no greater than the Series or Class Number (e.g. Series 1100 must not be tested to a pressure greater than 1100 kPa). Polyethylene pipe must not be tested to a pressure greater than 1.5 times the series number.

The leakage standard irrespective of pipe type used by the Ministry of the Environment is 2.22L per day per mm of internal diameter of pipe per km of pipe.

The test method is as follows:

1. Make sure that all anchors and thrust blocking has been installed in the pipe.

2. Install a pressure pump with make-up reservoir in the system to be tested. It should be installed at the end of the pipe, at a service connection or at a hydrant.

3. Start filling the test section with water.

4. Start applying pressure with the pump. Before bringing the pressure to the full test value, bleed all air out of the mains by venting through hydrants, service connections and air release valves.

5. Once the lines are full and all air has been let off, apply the full test pressure of 1 MPa.

6. After the test pressure has been maintained for several hours, conduct a leakage test by measuring with the make-up reservoir the amount of water that has to be pumped into the line in order to maintain the test pressure at 1 MPa for one hour.

7. Measure the amount of leakage and compare it to the standard of 2.22L per day per mm of diameter per km of pipe.

8. If less than the allowable leakage has been measured the line is satisfactory.

If the line has failed the leakage test, it is necessary to find the point or points where the excessive leakage is taking place. Before rushing off to dig up the entire line, there are a few steps that can be taken to ensure that it is a pipe leak, and not a leak at a fitting. First, leave the line under normal pressure. The next day, repeat the test. If the leakage measured the next day is greater than before, the leak probably is in a pipe joint or a damaged pipe. If the leakage is the same, it is more probably in a valve or a service connection.

To determine which it is, take the following steps. Insert the key for the curb stops in each shut-off, and listen at the top of the key. It may be possible to hear a leak since the key acts somewhat like a stethoscope. If a leak is heard, open the shut-off and close it again. If no leaking curb stops are found, crack the main valves at the ends of the test section several times and close them again. This is to flush out any sand grains in the valve seats that may prevent the valves from closing completely causing slight leakages.

If it is found that the leakage does not occur at either of the above points it is them necessary to try and find a leak through trial and error. Some sort of leak detector such as a sensitive microphone with amplifier and ear phones is necessary.

Any leaks in the line should be repaired, and the line re-tested until the measured leakage is less than the allowable leakage.

TESTING OF SEWER FORCEMAINS

Sewage forcemains are subjected to both pressure and leakage testing. The method of test is the same as outlined for watermains except that a lower test pressure is used. The test pressure is usually given in the contract documents and varies depending on the pressure used in the forcemain.

SUBJECT:

CONSTRUCTION OF SEWERS WATERMAINS AND ASSOCIATED FACILITIES

TOPIC: 21

RESTORATION

OBJECTIVES:

The trainee will be able to:

1. Understand the responsibilities which the Inspector has in restoration both in a surface related and a public related sense.

2. Recall which items of the total project which are considered restoration and how they should be handled.

RESTORATION

Many years ago, restoration was minimal. Poor compaction, a little bit of stone and a surfacing of emulsion was acceptable for a roadway. Persons fronting the jobs or the maintenance division of the local Works Department were expected to look after ditch and grass restoration and the continuing maintenance or filling in of the slowly settling trenches. Trees were ripped out or the roots were cut causing lingering death. They were seldom replaced or properly pruned and fed. Stream beds were silted in and littered with construction debris, which, hopefully was carried away with the next storm. The Inspector cannot allow this to happen, as the standard of restoration, in the minds of the area politician and the ratepayers, determines whether the job has been successful or has been concluded unsatisfactorily. The contractor, inspector, consultant and agency involved will be informed, in no uncertain words, if the job appears unsuccessful. Good restoration is the "icing on the cake" of any construction job and it reflects on the personal reputation of the Project Supervisor, Inspector and the Contractor.

General

Because restoration generally is done at the end or finalizing of a contract and, at that time, contractors are in a hurry to finish and go about other business, it is quite often given second grade status in the contractor's consideration.

To ensure that restoration is effected to an acceptable degree on MOE projects the requirements of Clause 21 of the Ministry of the Environment General Conditions (May 1976) are enforced.

Clause 21 states:

"21. Restoration

Where the Contractor enters into the lands or buildings of the Province or of any municipality or of any person or enters into any highway or road under the jurisdiction and control of any public authority for the purpose of making any survey, examination, investigation, inspection or other arrangements or lays any pipes or appurtenances in, upon, through, over or under any highway or road under the jurisdiction and control of any public authority, and in so doing disturbs any such lands, building, highways or roads, such lands, building, highways or roads shall be restored to their original condition without unnecessary delay."

This condition is considered minimal and in most cases the Contract Document improves upon the above.

Inspector Responsibilities

The construction inspector, when considering restoration, should do two things before starting the job:

(1) Make a personal inspection of the site with the Contractor and take pictorial records of the entire project. These will establish the basic standard for restoration of the site. Take additional pictures and discuss with the Contractor any areas which may become a point of contention. A good example of this is a sidewalk which is already badly cracked. Any problems such as this should be reported to the project supervisor in order that the contractor not be unfairly forced to replace that which he did not destroy.

(2) Carefully examine the restoration clauses in the contract specifications. They should be strict and should spell out exactly what the contractor is expected to do. After examining the site and specifications, make sure that both you and the contractor understand what is expected. In the event of discrepancies, they should be brought to the Project Supervisor's attention immediately for a decision. Remember, the Inspector is the person who is going to be talking to the irate homeowner who finds his prize begonias flattened, and who also may be paying for a sizable portion, in his taxes, for a job which he didn't want anyway.

There are many acts and local by-laws and policies which involve restoration. For example, most projects approved under the Environmental Assessment Act requires site restoration. The City of Toronto requires only temporary pavement repair; their crews do the permanent repair and then charge the cost to the contractor. The Town of Richmond Hill has a very stringent tree by-law which does not accept a one for one replacement on a mature tree which is on the road allowance and destroyed by a negligent contractor. They insist upon payment under a method of evaluation which could amount to several thousand dollars per tree. The inspector should make every effort to become familiar with these statutes, by-laws, and policies which pertain to the area in which he is working.

Restoration Considerations

Assuming that the job is started and on its way to completion, restoration includes the following items:

1. Backfilling of trenches. Depending upon the location, these may be backfilled with granular or native material and compacted to a specific Proctor density. If they are not properly compacted, they may settle at any time from the next rainstorm to many years later. Sagging trench cuts are one of the major nuisances faced by maintenance departments. Many bitter words are passed back and forth by the workmen about the ancestry of the inspector involved in the restoration which they are again restoring.

2. Asphalt or hard surfacing. It is best to save all hard surfacing work to the very end of the job. In fact, many municipalities request temporary repairs with cold patch and wait at least six months before performing a permanent repair. This gives the trench a chance to settle. Care should be taken during construction that all road surfaces be cut vertical to the horizontal plane and in as straight a line with the trench as possible. This saves recutting and widening of the area to be restored.

3. Sodding or spraying of grass. A good grade of sod or grass should be used. This should be placed over a thin layer of topsoil. Sod is usually used for lawn and boulevard restoration in urban areas. It must be constantly watered until there has been a reasonable catch, usually after about a month. Grass seed is sprayed on as a mulch in more rural areas such as fields and ditches. This does not require watering but is slower than grass to take and may require respraying of spotty areas at a later date.

4. Ditches and Culverts. These always seem to be ignored. Ditches are destroyed and only marginal efforts are made to bring them back to original shape. Culverts are allowed to fill in with silt or construction debris or have their ends bashed about by construction equipment and then just left. Possibly this is because they are in more rural areas. Check the ditches including culverts in the area to make sure they are still working properly. Remember, a plugged ditch in a heavy storm could flood out the construction site.

5. Trees and Shrubs. Damaged roots on a tree could kill it up to five years after construction. When a tree is damaged, it must be cared for. Damaged roots means the tree must be fed for several years and the limbs must be cut back. Damaged bark allows the tree to fall prey to insect, disease and fungus, any of which could bring its eventual downfall. A qualified expert must be brought in to take care of any damage done to trees. An amateur might only aggravate the situation.

6. Utilities. These must be properly located prior to construction, supported during construction and in the event of any damage will have to be properly restored.

7. Curbs, gutters and sidewalks. As previously stated, make sure that these items are not broken before construction. When reconstructed, they should match as nearly as possible the existing curbs and sidewalks, e.g. the texture finish of the new walk should match the old walk.

8. Machinery and Office Sites. Oil and gasoline must be cleaned up, all construction structures removed, the ground graded, top soil added and brought back to its original grade and condition.

9. Watercourses and Slopes. This has been covered in Topic 5, Environmental Considerations. Slopes which have been destroyed must be structurally restored, possibly with sod and rip-rap. Stream beds silted in with mud and debris must be cleaned out. Check the condition of stream beds a considerable distance down stream as debris is carried far from the actual construction site.

10. Roadway clean-up. Any construction will track dirt and dust over the various roads which the trucks are using. During wet weather, arrangements should be made to properly wash the mud from the wheels of the trucks at the site before it is tracked throughout the municipality. During dry weather, dust is formed upon the roads. It must be swept or shovelled from hard surface roads and graded to the shoulders or be treated with calcium on gravel roads. Do no place calcium on the dust on hard surfaced roads as it creates a muddy slick and could be particularly dangerous for cars braking and turning.

11. Traffic. A plan of the proposed traffic diversions should be arranged before construction starts. Prewarning signs should be placed in advance of construction and semi-permanent diversion signs placed with sacking over them ready to be removed on the day of "Start-up". When necessary, temporary roads should be built. When construction is complete, restoration involves removing these signs and filling in the holes, removing the temporary roads and putting the area back in original shape by grading, sodding, etc.

12. Garbage and snow plowing. When working on streets, these services must be maintained and yet there is seldom any specific mention of arrangements in the contract specifications. The wise Inspector, if he suspects there may be any problems with vehicular travel on the street, should check out arrangements.

13. Miscellaneous. All debris along the construction site must be removed; private driveways, walks, fences, lawns and private appurtenances put back in shape; the area swept up and given a good polish.

The weather plays a major part in restoration. Sod and other plants are best put in during the Spring and Fall. Asphalt and concrete is best placed during the warm weather. Very cold weather precludes much restoration other than granular backfill and debris clean-up.

SUBJECT:

CONSTRUCTION OF SEWERS
WATERMAINS AND ASSOCIATED
FACILITIES

TOPIC: 22

SEWER REHABILITATION

OBJECTIVES:

The trainee will be able to:

1. Be aware of and have a basic understanding of the different methods of Sewer Rehabilitation.

2. Identify and recognize the various deficiencies that might lead to sewer distress.

3. Be familiar with sewer inspection requirements prior to, during and following rehabilitation.

SEWER REHABILITATION

INTRODUCTION

As municipal sewer systems become older and more extensive it becomes increasingly more important to monitor and maintain them in a proper state of repair. When sewers are identified for rehabilitation, several methods of undertaking the work are available. The cost effectiveness of each method is considered together with other advantages or disadvantages, given the nature and location of the specific sewer deficiency.

The following will provide an outline of the various methods of sewer rehabilitation.

First, however, we will summarize the major typical faults or defects in a sewer system that would make it a candidate for rehabilitation.

SEWER DISTRESS

Sewer distress can be categorized generally as the following:

- Infiltration
- Structural damage
- Collapse
- Root penetration
- Surcharging
- Chemical attack
- Settlement
- Poor initial construction
- Leaking joints
- Leaking laterals
- Manhole deficiencies

One may suspect that a sewer line is experiencing distress if one or more of the following signs is present.

- Overflowing manholes
- Sewer backup into buildings
- Surcharged sewer
- Sunken areas above sewers or around manholes
- Increased flows

However, some problems may remain undetected unless a detailed sewer inspection is carried out.

SEWER INSPECTION

A sewer line or system may be inspected visually or with the use of closed circuit television.

Internal inspection requires climbing down manholes. Remember the hazards:

- Atmospheric hazards (Noxious Fumes)
- Physical injury
- Infection
- Insects and animals
- Exposure to potentially toxic materials
- Chance of drowning

At all times, strick adherence to Confined Space Entry and other Safety Standards is mandatory.

Internal inspection is limited to the inside of the manhole and the length of sewer that can be seen from inside the manhole using a light or mirror. In smaller sized sewers benching may prevent inspection of the sewer line. Using lights and mirrors, one may only be able to help detect sewers that are out of line vertically and/or horizontally.

A full inspection deficiency list should include the following:

Sewer Lines	Manholes
Types and size of pipe	Type of material
Horizontal & vertical deflections	Condition of walls
Joint separation	Condition of joint between
Broken joints	Manhole and frame
Root penetration in joints	Condition of benching
Circumferential or longitudinal	Condition of joints of pipe
Cracks	Connections
Crushed pipe	Any visible leakage
Protruding services	Condition of frame and cover
Broken services	Condition of steps
Leaking joints	Condition of pavement around manhole
Holes in pipe	Condition of riser rings
Corrosion of pipe walls	Condition of riser brick work
Calcite buildup	

When a visual inspection provides insufficient or inconclusive findings, a T.V. inspection should be commissioned. The television inspection technique has been well refined for determining joint conditions, structural deficiencies, sediment and sludge in the lines, root intrusions and lateral protrusions. Records are kept in log form and are usually accompanied by videotape or still photography for final report purposes.

The procedure uses a special waterproof camera moved through the line at a moderate (usually 10 metre/minute) speed to allow proper documentation of the conditions encountered. The movement of the camera is controlled with manual or power winches.

The monitoring is and must be continuous so that the camera may be stopped (or slowed down) at points of interest and/or retrieved if further progress is physically hampered.

One of the most important pieces of data collected is the distance from the starting manhole to the defect area. This is crucial, particularly is it is later deemed that excavation from the surface is required to repair the defect. Records of the exact location of each defect are based on these distances.

METHODS OF SEWER REHABILITATION

Once the extent of the sewer distress is fully documented, a number of sewer rehabilitation alternatives are available for consideration. The alternatives should be reviewed with respect to the following parameters:

- Cost
- Structural stability of the pipe
- Traffic disruption
- Disruption of public utilities
- Maintaining flows
- Safety
- Can the line be rehabilitated?
- Long-term capacity requirements (should the pipe diameter be increased?)
- Long-term effectiveness of rehabilitation

The order and significance of these parameters will vary from one jurisdiction to another.

(i) Excavation and Replacement

Complete replacement is time consuming, bothersome to the public, disrupts traffic and can be costly (mainly due to road and landscape restoration).

Sewer replacement is undertaken when manholes and pipes have lost their structural stability due to increased overburden loading, chemical attack or root damage. Another reason for sewer replacement is increased demand which eventually renders a system undersized.

Lines may also have to be reconstructed due to change in vertical or horizontal alignment due to bedding failures caused by poor initial workmanship or ground conditions. Bedding that has washed into pipe causes voids under and around a pipe or manhole causing large settlements. Settlements of manholes and sewer lines are also caused by chemical attack on the bedding, deteriorating it to such an extent that it can no longer support the structure it was intended for.

Structural instability involves pipes that are damaged so severely that pieces of pipe are missing, have been crushed or collapsed, or have large longitudinal cracks.

Sewer replacement provides a good opportunity to correct improper lines and grades, repair faulty connections, and eliminate stormwater connections from catch basins and roof drains. One big advantage is it replaces outdated pipe with up to date construction materials which will have the full expected life span.

Prior to replacement, existing capacity and future capacity requirements should be evaluated.

(ii) **Root Control and Removal**

Root intrusion is one of the major problems municipalities face today. It is mainly found in shallow lines and dry climates.

Sewer lines below the watertable do not normally have the problem of root intrusions. Two methods of root removal are mechanical and chemical. Mechanical methods are only a solution for the short term and appear to make roots more of a problem in the longrun. Chemical action will eliminate the root for a longer period and protection of the environment and treatment facility will control the chemicals permitted for use.

If roots are able to enter the system, it is through some kind of opening. The opening could be in the joint, or through a defective or damaged pipe. Therefore, a permanent physical repair or relining will provide the best remedy for the problem. But, if a line is severely blocked or has been damaged due to expansion during root growth, the section may have to be replaced.

Roots usually attack a sewer from the top and work their way down to the water source. Once the roots reach the sewage and are immersed in it, the submerged portion dies due to lack of oxygen. The root system above the flow continues to grow due to the high nutrient moisture content within the sewer. Root growth, left unchecked, could eventually block the flow.

Certain types of chemicals are absorbed by the roots and surrounding soil in the joint allowing the chemical and its inhibitor to be effective for a couple of years. Fumigants can be discharged into the system in the form of foam. The flow is not affected and actually has a function in this treatment in that it forces the foam up into the cracks and intrusions where roots may have entered. The flow will not immediately wash the foam away and it will cling to the sewer long enough for the roots to absorb the fumigant.

After chemical treatment, the leak should receive an application of grout or sealing compound. Complete biological disintegration of the root can take up to six months after application. After chemical or mechanical treatment, the line should be cleaned with hydraulic cleaning tools. Flushing of the line after chemical treatment should be postponed as long as possible to allow the chemical treatment to be effective.

Some of the more obvious reasons for controlling root growth are:

- To prevent blockage
- To restore full capacity and self-cleaning of the pipe
- To prevent damage to the pipe
- To prepare the line for cleaning prior to inspections and rehabilitation such as slip-lining, chemical grouting and inversion lining.

(iii) Chemical Grouting

A commonly used method of sewer rehabilitation is sewer pipe joint sealing done by pressure injecting a chemical grout into a joint or opening from within the pipe (packer method).

Some advantages are that it only takes a few hours for the operation, depending on the number of grouting operations. Utilities such as gas, Bell, cable T.V., power and other existing lines are not disturbed since all work is done internally and requires no excavation. Since grouting will not increase the structure strength, it cannot be done when sections of pipe are missing, crushed or collapsed.

Dehydration is a factor that affects the grout's life and effectiveness. If the moisture of the surrounding soil decreases significantly, it may cause shrinkage of the grout. The return of moisture at a later date may rehydrate the grout, but the grout may not completely re-seal the hole, and the grout or pieces of grout could be washed away by infiltration. The grouting operation also leaves a ring of grout around the joint and any residue grout may have to be flushed out of the line.

Surrounding soil conditions will have an influence on whether a joint can be grouted or not. The best conditions are where pipes are bedded in a fine sand. Soils with large voids present cannot be grouted as the pressure applied to the grout will continue to fill the voids in the soil and not the problem area. Also, badly aligned or offset joints are difficult to grout because the grouting device cannot be properly sealed to the inside wall of the pipe. Grouting devices can compensate for some roughness of a pipe, however, badly eroded concrete due to chemical action, cavitation etc., causes problems in acquiring a tight fit for the packer.

Sealing sewers by the packer method was first attempted in the early 1960's. With the introduction of closed circuit T.V., the camera/packer method became a more viable method to repair infiltration problems. Packers and cameras can be used in lines as small as 150mm.

There are two general types of grouts in use: gel and foam. The gel types of grout are low in viscosity, and as they are forced out through the opening or joint, they displace any groundwater and mix with the surrounding soil. Within minutes after the gel has been injected, gelation occurs forming an impervious plug around the opening.

The foam types are high in viscosity prior to being mixed with water. The urethane foam viscosity decreases after mixing with water. As it foams and expands, the viscosity then increases until it has reached maximum expansion or it has filled all the voids it was intended to.

For the proper use of the packer method, a clean, smooth and non-porous pipe surface is required. Be cautious of any condition which may prevent the inflatable end elements (sleeves) from making continuous contact with the inside of the pipe.

The procedure for Chemical Grouting is as follows:

1. Packer is centered over infiltration location, ensuring end sleeves are not over opening to be grouted.

2. Sleeves are inflated. Sleeves at this stage are touching, but not pressing against the pipe and therefore, the void will not hold a pressure.

3. Upon fully inflating the sleeves, the void should be able to hold a 70 kPa (10 p.s.i.) pressure with no loss. A test should be conducted on a clean section with no opening or joint present.

4. Monitor void pressure.

5. Mixing of grout takes place either in the void or as it is injected into the void. The two part grout reacts chemically upon mixing, either forming a gel or foam to seal the joint.

6. Grout may blow past the end sleeves.

7. After the grout is turned off, a waiting or curing time of anywhere from 30 seconds to 2 minutes is required depending on the type, mix ratio, additives and temperature. Pressure should hold at 10 p.s.i.

8. The packer is deflated.

(iv) **Slip-Lining**

Slip-Lining involves sliding a flexible liner pipe of slightly smaller diameter into an existing pipeline. This is followed by reconnecting the service laterals to the new liner.

Polyethylene has been a common material used for slip-lining pipelines. Polyethylene pipe used for slip-lining is currently available in nominal diameters as small as 100mm.

Slip-lining is used to rehabilitate extensively cracked sewer pipelines, deteriorating pipe installed in a corrosive environment, pipes with root intrusion problems, and pipelines with relatively flat grades. Since the size of the liner pipe is smaller than the existing sewer, the use of this method is limited by capacity requirements.

One of the advantages of relining rather than replacing a sewermain is that it requires minimal excavation. This limits traffic disruption and interference with surface structures. The insertion of a pipe with heat-fused joints will also help eliminate problems with root intrusion and joint infiltration. Slip-lining can be used in pipelines having moderate horizontal or vertical deflections.

Before installing a liner pipe, the existing sewer should first be inspected by closed circuit TV to identify all obstructions such as displaced joints, crushed pipe or protruding service laterals. The inspection should locate all lateral connections that are to be connected to the new liner pipe.

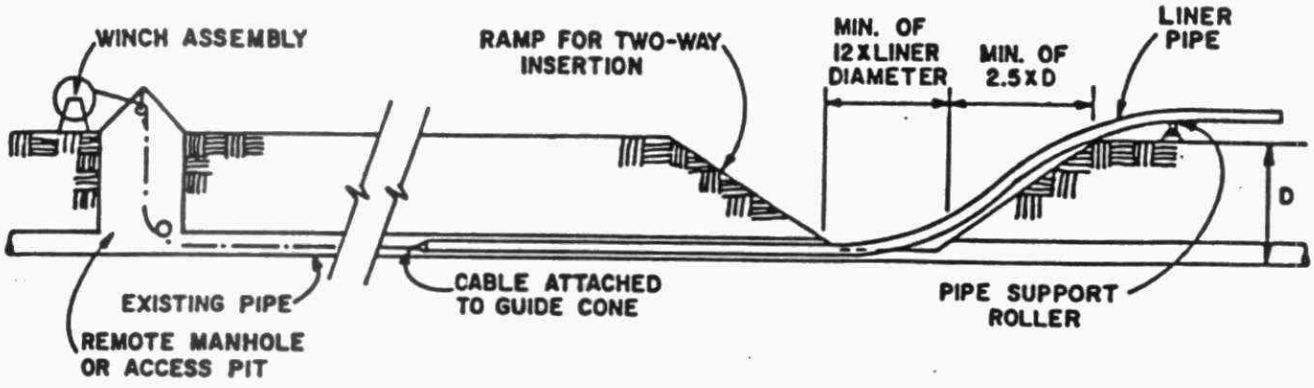
The liner pipe can be pulled or pushed through an existing sewer. The method chosen is partially dependent on the type of pipe to be inserted and the type of joint. Pulling is the most common method used and requires excavating a pit at one end of the section to be lined.

To pull a liner pipe through an existing sewer, a steel cable is threaded through the existing pipe and is attached to a pulling head which guides the pipe end past minor obstructions. Sections of line pipe that have been butt-fused together in the field to form a flexible continuous pipe are then pulled through the sewer by use of a winch.

The pulling operation tends to stretch the line pipe. Excessive stretching should be avoided. Once the lining has stabilized in length, the connections to manholes and the re-opening of service connections follows.

Wastewater flow in the existing sewer may or may not need to be interrupted during insertion of the slip-lining. Depending on the size of the insertion pipe, the flow may continue in the annular space between the two pipes. If not, it may be necessary to temporarily plug the upstream lines and using above-ground piping, pump the flow around the section being lined.

Slip-lining Installation



(v) Inversion (Insituform) Lining

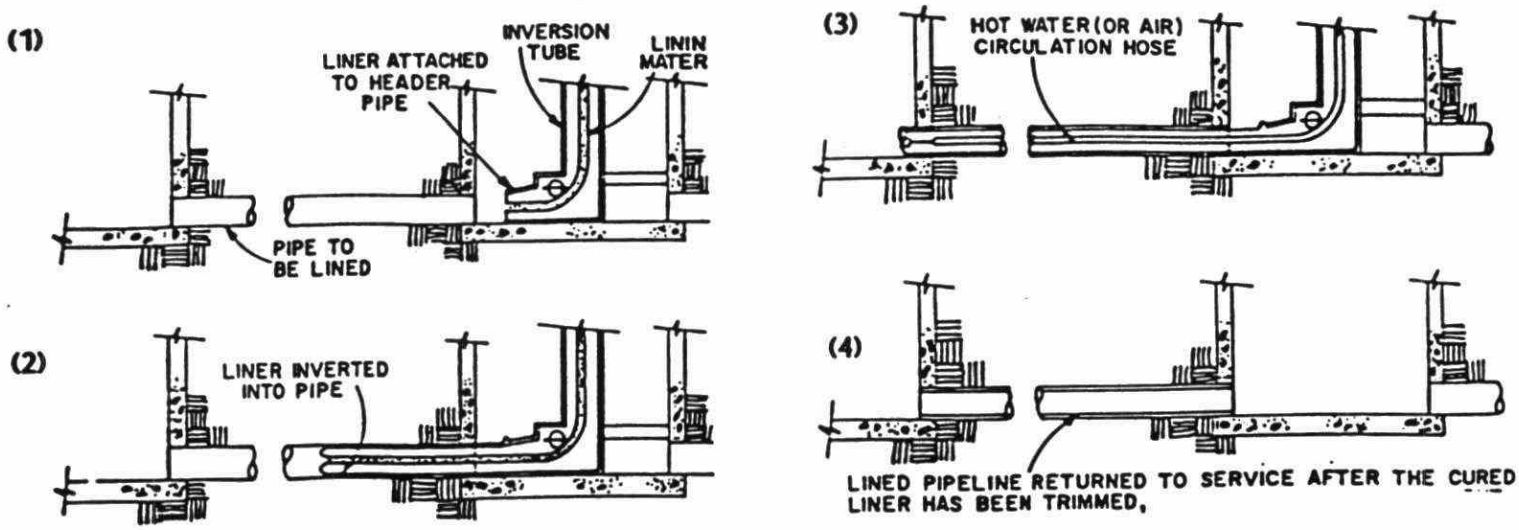
The Inversion (Insituform) Lining system uses thermally hardened flexible liners installed from the ground surface using existing manholes for access. After the lining system has been installed and cured, a cutting device is used with a closed circuit camera to reopen the service connections.

The liner is installed through a large tube that is placed into a manhole. At the bottom of the tube is an elbow that guides the liner into the pipe to be lined. The liner is inserted into the tube and is attached to the end of the elbow. Then, water or air is pumped into the tube. As the pressure builds, the flexible fabric is pushed through the pipe and is inverted into place. The method by which this material is installed allows greater flexibility in changes in grade and alignment than the slip-lining method.

With the liner in place, the water or air is heated thus causing the thermosetting resins in the material to cure and harden. Following curing and cool-down, the water is drained away and the ends of the liner are cut and sealed at the two manholes.

Because the liner expands to fit the existing pipe geometry, Insituform Lining is applicable to egg-shaped or arched pipelines. This method reportedly is effective for resolving corrosion problems. Inversion lining is also applicable to sewers needing minor structural reinforcement.

Inversion Lining Installation



There are numerous other relining systems now available. These include other types of polyethylene and PVC, either deformed, folded or reduced in size in some manner for initial insertion. There are segmented plastic pipe lining systems as well as new techniques involving spiral-wound pipe lining. Many of these systems are relatively new and information on long term results is not yet available. The two lining systems discussed have been in use throughout Europe and North America for over twenty years and have a significant established performance record. Other systems including segmented and deformed pipe have been in service successfully for 10-15 years and can be considered with the same degree of confidence as the older systems. Remember that many conditions may determine which type of lining system should be used.

There are also less common specialized systems which include pipe bursting, mechanical sealing devices and various applied coatings and cured type liquid media liners.

SUMMARY

It is important to remember that any rehabilitation work on sewers is intended to prevent further deterioration and provide an increased and predictable life expectancy. The evaluation process to determine rehabilitation need and cost benefit is extensive and must include input from maintenance staff.

In some cases, circumstances may dictate that nothing be done until further deterioration takes place. Full replacement would be scheduled then.

Scheduled system monitoring is the best means to ensure predictability and enable prudent planning and budgeting. Appropriate record keeping will provide documentation necessary to support rehabilitation as well as replacement construction.

SUBJECT:

CONSTRUCTION OF SEWERS
WATERMAINS AND ASSOCIATED
FACILITIES

TOPIC: 23

WATERMAIN
REHABILITATION

OBJECTIVES:

The trainee will be able to:

1. Be aware of and have a basic understanding of the different methods of Watermain Rehabilitation.

2. Identify and recognize the various deficiencies that might lead to watermain distress.

3. Be familiar with watermain inspection requirements prior to, during and following rehabilitation.

WATERMAIN REHABILITATION

INTRODUCTION

Rehabilitation is a process that restores the capacity and reliability of conveyance systems. this procedure is now becoming quite common in Ontario. Before engaging in the restoration of watermain systems, one must consider the type of problem and the cost effectiveness of refurbishing as opposed to a decision to replace the system. The pros and cons of both rehabilitation and replacement must be carefully assessed.

This chapter discusses the various methods of rehabilitation in Ontario and concludes with certain comments on watermain replacement.

The chemistry of treated drinking water varies considerably from system to system. In addition, the water quality may have an adverse effect on the interior wall of the conveying pipes if the walls are not adequately protected.

The effects mainly are:

1. Corrosion
2. Encrustation on pipe walls

Velocity expressed in imperial units in pipes is determined by the Hazen Williams formula:

$$V = 1.318\ C\ r^{0.63}\ S^{0.54}$$

where C = coefficient of friction or a measure of the roughness of the pipe interior
r = hydraulic radius
s = slope of the energy gradient line

Table 1-1

Condition	C
New Pipe	130-140
Fair to normal (interior clean)	100
Significant reduction in pipe capacity	70
Severe problem-interior cross section greatly reduced	30-50

With reference to Table 1-1 it should be noted that velocity, and therefore flow, is adversely affected by the interior condition of the pipe.

It should be noted that flow conditions in watermains can also be affected by sediment in the water and the fouling of pipe lines.

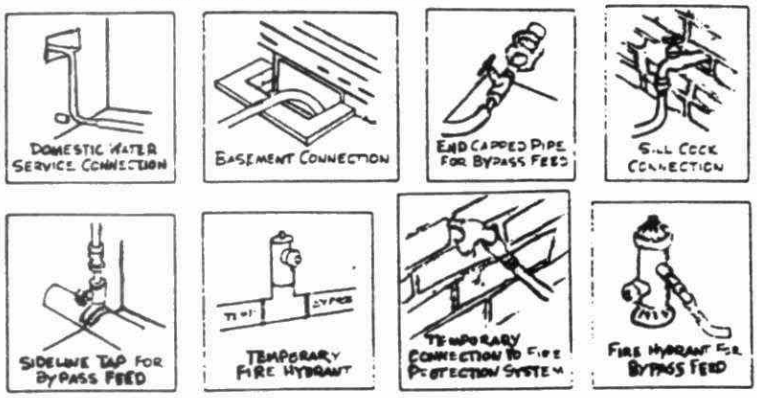
To overcome the flow restriction in the conveyance system or to rehabilitate the conveyance system, the methods used are:

1. Periodic cleaning
2. Lining the conveyance system
3. Replacing the existing system

In the case of watermains, the continuity of service to the consumer is important.

This continuity may be maintained by an above ground bypass distribution system to serve customers affected by the rehabilitation procedure. 50mm, 100mm, 150mm or larger pipe should be provided to ensure proper domestic service and fire protection. Bypass lines should be buried where necessary to allow for pedestrian and vehicular traffic.

TYPICAL BY-PASS TECHNIQUES


DOMESTIC WATER SERVICE CONNECTION
BASEMENT CONNECTION
END CAPPED PIPE FOR BYPASS FEED
SILL COCK CONNECTION
SIDELINE TAP FOR BYPASS FEED
TEMPORARY FIRE HYDRANT
TEMPORARY CONNECTION TO FIRE PROTECTION SYSTEM
FIRE HYDRANT FOR BYPASS FEED

REHABILITATION - BY CLEANING

Watermains can be cleaned in place by using various methods such as:

- hydraulic
- drag cleaning
- drag mechanical
- water jet
- hand methods

With hydraulic method, a foam swab is pushed through the line by water pressure and, the cleanings are discharged into a receptacle and removed from the site. This method is used where volume and system pressure is available.

When hydraulic cleaning is not practical, the drag technique is employed. With this method, the cleaning device is winched back and forth through the lines, until they are suitably clean. In some instances, this method is water assisted to flush out debris as the cleaner scrapes the pipe wall clean.

REHABILITATION - BY RELINING

In general, lining protects the interior of the pipe from deterioration due to the chemical reaction between the water and pipe material. Different linings are required for different water qualities. Various lining methods commonly used are:

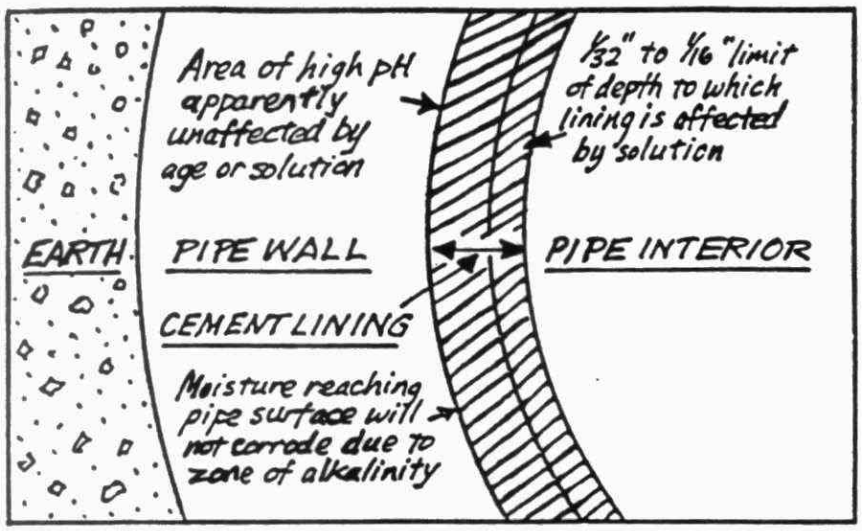
Cement Lining:

This method is used with iron based pipes such as unlined cast or ductile iron and protects the pipe through two properties of the cement lining itself. First, there is the alkalinity of the mortar and, secondly, there is the gradual decrease in the amount of water in contact with the pipe wall.

The highly alkaline condition at the pipe wall slows the corrosion process and eventually becomes static since the chemical reaction of the cement, air and water tend to clog the porous cement, therefore, the lining provides both a physical as well as a chemical barrier.

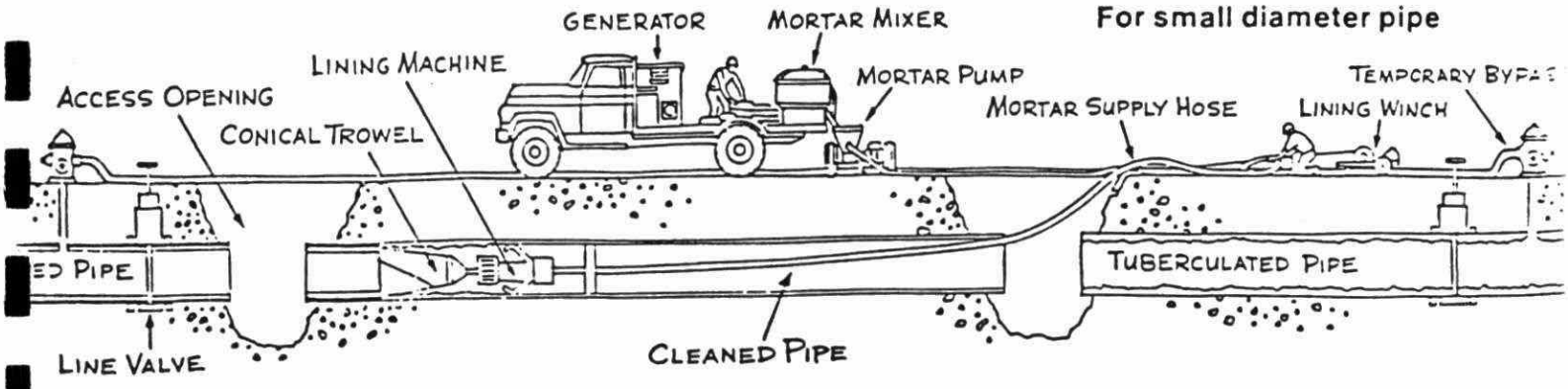
The lining stays in place due to the arch action inherent in the shape of the cement lining. Once the lining has hardened, it forms a continuous arch - a pipe within a pipe.

ZONE OF ALKALINITY IN CEMENT-MORTAR LINING


Area of high pH apparently unaffected by age or solution
1/32" to 1/16" limit of depth to which lining is affected by solution
EARTH
PIPE WALL
PIPE INTERIOR
CEMENT LINING
Moisture reaching pipe surface will not corrode due to zone of alkalinity

The method of lining is as follows: a mortar consisting of a 1 to 1 mix of sand and cement is conveyed either by mechanical vehicle or a constant displacement pump to the lining machine. The mortar is extruded through finger plates on the lining machine, picked up by the rapidly rotating head and centrifugally discharged onto the pipe wall. The thickness of the lining is governed by the speed of the lining machine as well as the extrusion rate of the mortar. Conical trowels for smaller diameter pipe and rotating trowels for larger diameter pipe finish the lined surface to ensure optimum hydraulic efficiency. Services are re-opened by back flushing with air or water before the mortar sets. Ends of the pipeline are immediately closed following lining to ensure proper curing.

This procedure results in a hydraulically efficient, long lasting pipeline at a lesser cost and without the inconvenience of replacement.



Slip-Lining

Insertion of flexible thermoplastic liners directly into the watermains is known as slip-lining. The most commonly used pipe material is polyethylene. It is used mainly by sewer and natural gas utilities, however, it is also used in potable and raw water transmission and distribution systems. The key benefit of slip-lining is that it creates a new integral pressure pipe inside the old, deficient pipeline without undertaking a complete excavation of the pipeline.

Using a process known as thermal butt fusion, the ends of several consecutive 13m lengths of flexible pipe are joined above ground at a convenient location to form a single length of pipe, about 30m long. One end of this pipe is then pulled by a cable into the entry pit through the section of the old pipe in a few minutes, then reconnected to the existing mains.

Chemical Lining

Chemical lining of pipelines includes those processes that involve the addition of chemicals to the water inside of the carrier pipe. Chemicals are added in order to obtain a lining on the pipe wall or a chemical coating that will inhibit corrosion or metal piping. Processes include chemical film, chemical coating, and inversion (insituform) lining.

Chemical Film

A chemical film of zinc orthophosphate, which is fed into distribution system water, inhibits corrosion on the interior wall of a metal pipe. It has been shown that this method can also be successful on nonmetallic piping systems, such as asbestos-cement pipe.

Zinc orthophosphate is fed into the system at a dosage recommended for each system. The feed mechanism is a chemical metering pump.

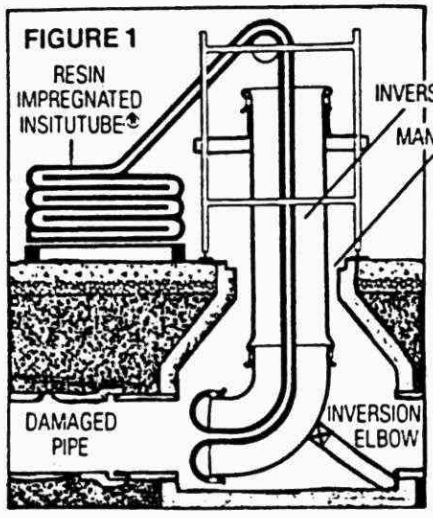
Chemical Coating

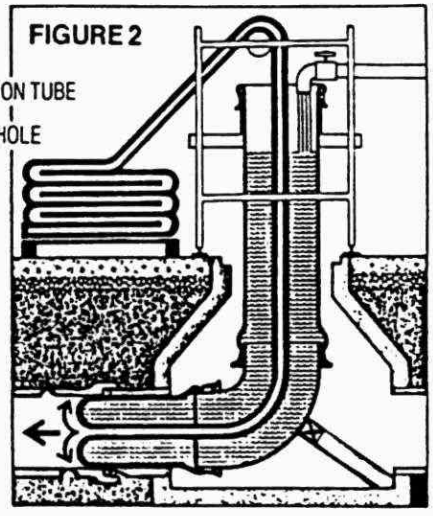
Colloidal calcium carbonate, or calcite, added to water that is fed into a main will form a chemical coating of calcium carbonate. This coating protects the interior walls of metal pipe and inhibits corrosion.

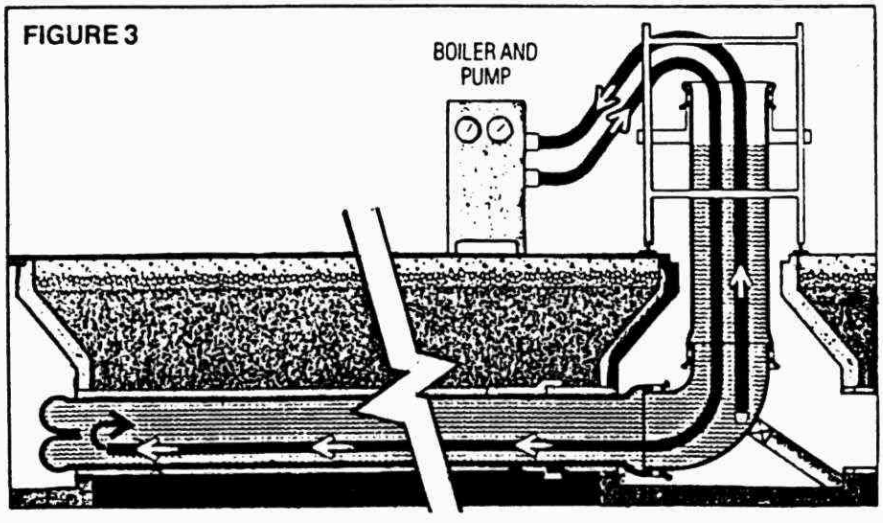
This method, sometimes called the calcite lining or McCauly process, requires that the main be cleaned first. The chemical can then be added to only one main, a group of mains, or to an entire section of the distribution system. One problem with this method is that a specified quantity of calcium carbonate must be continually added to the distribution system at the treatment plant or directly into the main in order to prevent the lining from dissipating. As a result, the lining is sometimes not evenly distributed, being more uniform close to the treatment plant and less uniform or non existent further out into the system.

Inversion (insituform) Lining

Inversion (Insituform) Lining originated in Great Britain in 1971, and was first brought to North America in 1977. The process uses a polyester-fibre felt tube, which is coated on one side with an impermeable material and impregnated with a liquid thermosetting resin on the other side. The tube is fed into the pipe to be lined by an inversion process (Figure 1), which is achieved by filling the tube-feeding standpipe with cold water (Figure 2). The tube is pressed tightly against the pipe wall when it is fully extended. Following extension, the water is heated to cure the resin and create a pipe inside of a pipe (Figure 3). It is recommended that the pipe be cleaned beforehand, although this process can be used without cleaning the pipe for repairing damaged or partially destroyed pipelines.







When first inverted into a pipeline, the tube is pliable and the resin is a viscous liquid. It is this characteristic that allows the tube to negotiate 90° bends and to fill cracks and holes. The reduction of the internal pipe diameter is similar to a coating being applied to the pipe, and the smoothness of the inner surface may enhance the hydraulic capacity of the line. This process has been used on pipelines with diameters ranging from 100mm to 2700mm.

CATHODIC PROTECTION

External corrosion is the wearing away of a substance from the surface inwards by an electrochemical process. In order for a corrosion cell to occur, there must be four components present: an anode, a cathode, a metallic path between the anode and cathode, and an electrolytic path between the anode and the cathode. If there is no difference in potential between the anode and the cathode, corrosion will not occur even though all four components are present. This, however, is rarely the case; even on a single small piece of metal, different points of the surface will exist at slightly different potentials. This is caused by factors such as the coating being scraped off or the length of pipe running through dissimilar soils.

One way to stop corrosion is by adding another metal to the existing corrosion cell, which is more electro-negative on the galvanic scale than the present anode, thus making the cathode more negative. The more negative the metal that is added, the more negative the cathode becomes until it is just as negative as the anode in the original corrosion cell. At this point, there is no more potential difference between the anode and the cathode and therefore corrosion will cease to occur.

The metals most commonly used for this purpose are magnesium and zinc. When connected to an existing watermain they will become the anode. As the anode is where the corrosion metal loss occurs, it will be consumed in providing protection to the watermain, hence, the term "sacrificial anode".

The number of anodes depends on how much current needs to be added to halt the corrosion. A rule of thumb which has emerged for iron pipe water distribution systems with copper water services is the addition of 1.0 milliamps of current per square foot of watermain pipe.

With the use of the cathodic protection method a number of problems have been brought under control.

REPLACEMENT OF WATERMAINS

The decision about whether to replace watermains is usually based on engineering judgement, however, one rule for watermain replacement is based on a comparison of replacement costs with the cost of breaks.

As pipes age, the breakage rate tends to increase, and the pipes become likely candidates for replacement. The replacement of a watermain should NOT be based only on the subjective judgement made by utilities engineering personnel. It should be decided after applying the following quantitative rules of determining when to replace watermain because of leakage, pipe breaks and inoperative valves.

These rules have been developed by Shaines and Howard. It is based on the fact that it is possible to develop an equation giving the total cost of pipe replacement, leakage, broken valves and pipe breakage.

The replacement rule boils down to a comparison of the cost of replacement with the sum of the costs of (1) pipe breaks, (2) water losses caused by leakage, (3) leak detection and repairs, and (4) valve replacement, which is used as an indication of the cost of broken valves. Thus if cost of replacement equals the cost of maintenance and repair its time to replace.

SERVICE CONNECTIONS

In various municipalities, the bulk of all complaints received about the water distribution system are often about dirty or rusty water and low water pressure. The first problem can be controlled by cleaning the watermain and cement lining the mains. The second problem can be investigated, and if often found to be a mineral build-up on the inside of the copper water service pipe. This build-up can generally be characterized by the common term of "Kettle-scale" which is a deposit of calcium and magnesium salts.

One community follows this procedure to restore flow and improve service:

a) Remove the water meter and sand-blast the service connection. The air and sand are forced into the waterline in the house.
b) When all the water services on the block are cleaned, turn the water in the main back on and flush the accumulated sand and dirt out of the watermain via a hydrant.
c) Flush any residual sand grains or dirt out of each service connection.
d) Install a new water meter.

To date, this cleaning process is still being used and has achieved a change in the average flow rate from 13 to 23 L/min.

This allows the carrying out of the Finance Department's meter change program much faster than would otherwise be the case. Depending on how the costs are assigned, this will, therefore, allow water service connection cleaning or the meter change-over to be accomplished at a much lower cost than originally anticipated.

Metering provides not only system accountability, but has been shown to reduce water use. It is one of the first operations performed as a result of Water conservation programs.

MOE
Standards Development Branch
LIBRARY

TD
678
.S49
1993

Sewer and watermain
construction inspectors course /
77701